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A YEAR'S WORK ON THE ALASKA CENTRAL RAILWAY.

By A. W. Swanitz.*

The project of the Alaska Central railway was conceived, about two years ago, by a number of prominent Northwestern men, under the advice of Mr. C. M. Anderson, C. E., of Seattle, who was more or less familiar with Central Alaska and the feasibility of obtaining a practical line into the interior. Government explorations and geological surveys, under Mendenhall and Major Glen, had confirmed Mr. Anderson's statement, not only as to the feasibility of the line, but also as to the vast agricultural and unsurpassed mineral resources of the country between Resurrection Bay, the Turnagain Arm country, the Matanuska, Sushitna, Chulitna and Cantwell River valleys and the Tanana River.

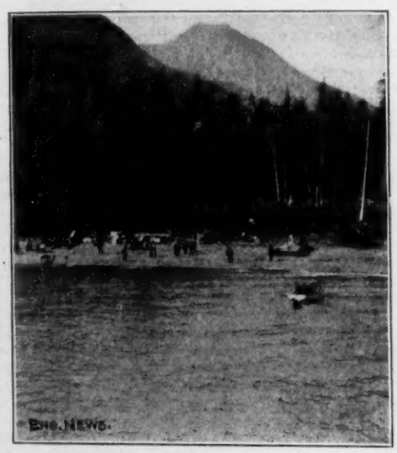


Fig. 2. First Landing of Construction Force at Resurrection Bay, Aug. 28, 1903.

The principal questions to be solved were, first, an open-all-the-year-round harbor on the Pacific Coast, grades practicable for economic hauling of ore and coal at a reasonable cost, and the governing passes of the Alaska Coast range and mountains east of Mt. McKinley. Accordingly eight engineer corps were placed in the field, under C. M. Anderson, Chief Engineer, two years ago, with instructions to obtain the solution to these questions by a preliminary survey from the Coast to the Tanana River. Their various reports fully confirmed previous impressions, showing a possible alinement almost due north from Resurrection Bay; showing an unsurpassed landlocked harbor at the southern terminus, and a crossing at Moose Pass, of the Coast Range, at a maximum elevation of 900 ft., and a crossing of the mountain range east of Mt. McKinley, at Broad Pass, at 2,300 ft. maximum elevation.

Again, their reports showed the wonderful resources of that country in placer and ledge gold fields; in coal of superior quality, assaying from 65% to 90% fixed carbon; in large outcroppings of copper ore; in tin and platinum on the upper reaches of the Sushitna River; in fertile valleys of grazing lands, and confirming the official reports of the Government agricultural stations showing that all of the vegetables of the northern temperate zone, grains and grasses were, and could be, raised in the interior valleys with equal success and results as in Michigan, Wisconsin and Minnesota.

Encouraged by these reports, further preliminary capital was raised and the first actual construction party was landed at Resurrection Bay, under the writer, in August, 1903. Fig. 2 shows the landing of the first construction party at the beach of Resurrection Bay. Nothing was there to encourage newcomers but a great forest of spruce, hemlock, etc. To-day, after nine months, there is a town of about 900 inhabitants, with stores, electric light and telephone systems and all the appurtenances of a Northwestern railway and mining town.

The original construction party, at landing, was provided with tools and supplies of all kinds, from a box of pills to a sawmill outfit; from calico and coffee to pile driving machinery. In short order the sawmill was set at work, and the first piles for the wharf were driven by Nov. 15 of last year. A small force of from forty to fifty men was kept at work, without interruption, every day of last winter. The daily record of temperature showed a minimum of — 1° F. on Feb. 6; an average of 30° F. in December, 28° in January, 17° in February and 30° in March, and showed a maximum snowfall of 24 ins.

Fig. 7. Construction Train No. 1 at Fish River Flat, Mile 4.

The construction force was largely increased by the last of March, and a substantial wharf and warehouse, with pile trestle approach to the shore, was completed in April. The first rails of the Alaska Central railway were laid on the wharf at Seward, the southern terminus, on April 16. Since

FIG. 3. STARTING THE FIRST SAWMILL, NOV. 10, 1903.

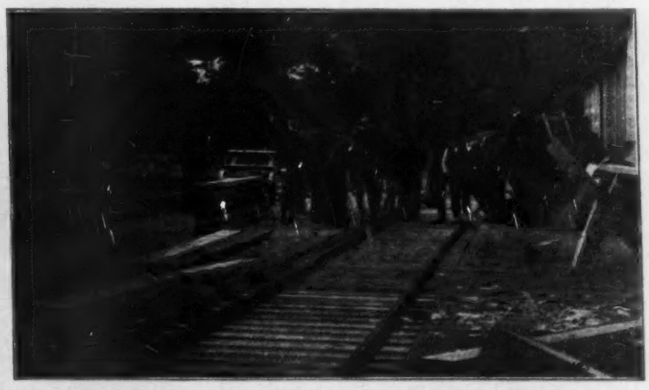


FIG. 5. LAYING FIRST RAILS OF THE ALASKA CENTRAL RY., APRIL 16, 1904.

*Chief Engineer Alaska Central Ry. Co., Seward, Alaska.

then, regular construction work has been pushed, under increasing pressure, with the result that at the date this is written, July 12, seven miles of track have been laid, the first eleven miles of grading have been completed and an additional twenty miles is under fair progress of construction, as illustrated herewith.

Considering the distance of this enterprise from the labor and supply market, its cost to date has

Fig. 1. Map Showing General Route of the Alaska Central Ry.

been very reasonable indeed and much below the original estimates. On the first division of 25 miles, twelve miles show an average per mile of 6,000 yds. earthwork, 80% grubbing and clearing; eight miles show an average of 10,000 yds. earthwork, 3,000 yds. rock and 85% grubbing and clearing; five miles show between 20,000 and 30,000 yds. earth and rock work and 90% grubbing and clearing. The same averages hold fairly well of the first one hundred miles; the second and third hundred miles are much lighter; while the last 113 miles are again heavier work.

About half the distance of the 413 miles passes through a heavily timbered country of spruce and hemlock, with some cedar, birch and Balm of Gilead trees. The other half traverses largely the grazing lands of the Sushitna and Chulitna valleys.

The heavier construction work consists principally of sidehill cuttings. The cost of labor has been firmly established, at this time, at $2.50 per day for all shovel work, grading and surfacing; at $2.75 for drilling and powder work; at $3.00 for bridge carpenters and steel men; and at from $3.50 to $4.00 per day for foremen. Comparatively few teams are used on grading. Teams for logging, tie and sawmill work are paid, with driver, $6.00 per day; the cost of feed averages 98 cts. per team per day; drivers' wages, $2.75; stable and harness repairs, about 21 cts. The cost of earthwork, under these circumstances, being nearly all pick, shovel and barrow work, has averaged to date about 17½ cts. per yd; solid rock work, 72 cts. per yd, counting 20 cts. per yd. for cost of powder and its transportation.

The character of the rock varies somewhat. Micaceous schist and slate seems to be prevailing. Auriferous quartz and limestone has been found in a number of cuts.

The cost of bridge work has been found much below the general average and running as low as $2.95 per lin. ft. on pile bridging. The company's sawmill furnishes 8 × 16 × 28 ft. stringers at $8 per M.; piles are 1c. to 1½c. per lin. ft. delivered, 6 × 8-in. × 8-ft. ties cost 25 cts. delivered on the right of way. There is a superabundance of heavy timber within easy reach at nearly all points of the line. The Alaska spruce, used for piling, has been found of excellent quality and piles 100 ft. long, 10 ins. at the small end, and showing scarcely any effect at last hammer blow with a 3,000-lb. hammer, are readily procurable.

The construction company is maintaining a large and efficient commissary, furnishing board to its employees at $6 per week, but no employe is prohibited from keeping his own mess, and many of the smaller station men and subcontractors claim that $4 per week covers all their expenses. The only labor difficulty and trouble experienced so far is caused by frequent findings of rich placer gold locations. A large number of prospectors are constantly at work on creeks and river tributaries within easy traveling distance. These men will come to Seward, the road's southern terminus, with nuggets and gold dust often showing net earnings of from $8 to $15 per day with rocker and pan, and generally, after a payday, a not inconsiderable portion of the grading and tracklaying forces scatters to try prospecting, in hopes of big finds. About one in twenty will succeed, in a week's time, in finding a good place, while the others straggle back, quite willing to resume the company's shovel and pick.

All things considered, there is a great future for this railway. It will place the Yukon River, Nome and Dawson and the great new mining district of the Tanana within five days easy travel of Seattle, Portland and Tacoma, as against 28 to 30 days now necessary for the trip. It gives easy access to central southwestern and southeastern Alaska all the year round, whereas, now, for five months of the year, the more remote regions of Alaska are inaccessible. It opens up what is undoubtedly the greatest storehouse of mineral wealth in the world and brings its resources within reach of everybody, while the country's marvelous scenic beauties and superb climate will unquestionably attract many sightseers, especially during the summer months.

A 16-MILE, 80,000-VOLT, EXPERIMENTAL ELECTRIC TRANSMISSION LINE.*

By A. C. Balch.

Early in 1903 a discussion arose regarding the proper insulators to be used on the then proposed Kern River transmission line, which was to be 125 miles in length, and to deliver at its terminal 10,000 HP. The copper had been determined for a working voltage of 45,000 at the power house, and 40,000 at Los Angeles with both lines working. The country about Kern River, however, having great water powers available, it was quickly seen that a higher voltage would be desirable in order to carry more power to Los Angeles over the same line. Accordingly, Mr. W. G. Kerckhoff, president of the Pacific Light & Power Co., instructed that a test should be made of the various types of line insulators then on the market.

Nine types of insulators were available for testing and each type was installed part with wooden pins and part with metal pins; the cross arms were painted with metallic paint.

For convenience the insulators here are referred to as "Porcelain 3-14-29½"; the first figure being the number of pieces, the second figure the diameter of the largest petticoat, and the third figure the creepage surface, i. e., the actual surface distance between the wire and the pin. Stated in this way the insulators were as follows:

Locke Porcelain 3-14-29½
Locke Porcelain 3-11-28
Locke Porcelain 3-11-27
Locke Porcelain 2-11-25
Locke Porcelain 4-14-35
Locke Glass 2-11-23
Provo Glass 1-9-16
Muncie Glass 1-9-16¾
Knowles Imperial Porcelain 1-10-21
Allen Special, composed of 2-story Provo 1-9-16, a single insulator being mounted on top of two others with a cross piece supporting the upper insulator.

On April 1 a line 16 miles in length, having three wires mounted in triangular shape with a distance of 40 ins. between wires, was started up. The test continued until June 1. During this time all sorts of weather was experienced, including heavy fogs and light and heavy rainfalls. In one night 4 ins. of rain fell on the country through which the line was built.

The line ran east and west through the San Gabriel Valley, about one mile from the foot of the Sierra Madre range and parallel to the same.

During this test the usual experience of having no difficulty whatever with new insulators was most prominently noted.

There was an average voltage of 80,000 between two of the wires and the third wire. Only single-phase transformers were available, and these were two 40,000-volt transformers, which were connected in series with the center between the two grounded. Each transformer had a capacity of 250 KW. Two of the wires were connected to one of the outside terminals of these transformers and one to the other. The current transmitted to the line was measured with a standard ammeter, which was placed in the line wires and set on top of the line insulator. This ammeter showed no apparent tendency to become inoperative on account of static charge. The current to the line was approximately 3 amperes, and this did not vary more than 5% at any time during the test.

The size of the wire was No. 4 B. & S. gage.

A sharp rattling sound was in evidence, which seemed to be localized at points along the wire, varying from one pole length to a half pole length apart, but could never be placed. It is difficult to determine whether the noise actually shifted or whether it was an error due to the listener. At times it seemed as though objects extending up from the ground, like neighboring trees and poles standing in the line, made greater noise apparent in their locality, but this was not definitely found to be the case. No danger could possibly have come to a lineman by his thinking the current was off when it was on, as the sound could easily have been heard a hundred yards from the line at any time. A native of the locality said that the line sounded like bees, and a great many of them. A reduction of 20% in the voltage practically stopped all the noise.

The line was luminous on a dark night or when looking toward a dark mountain on any night, and presented the appearance of a stream of bluish light about 3 ins. in diameter for each line. This line did not show any nodes or extra brilliant points, but seemed to have exactly the same luminosity throughout its entire length. There was no extra intensity of brilliancy at the insulators or any other place, except at the tie wire on the top of glass insulators it seemed to somewhat enlarge the luminous volume which appeared as a somewhat brighter point than the other portions of the wire. The single wire, however, was considerably brighter than the other two, and could

*A paper presented before the Pacific Coast Electric Transmission Association at Monterey, Cal., June, 1904.

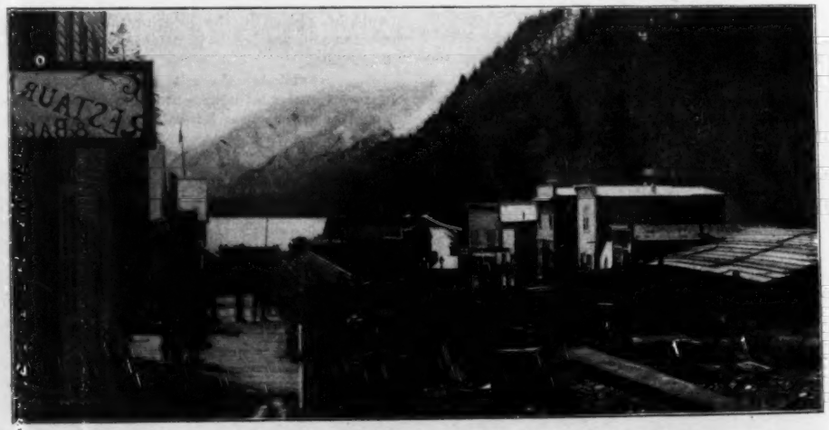


FIG. 4. LOOKING DOWN FOURTH AVENUE TOWARD THE PIER, SEWARD, ALASKA, MARCH 28, 1904.

be seen at times and conditions of darkness when the other two wires were not in evidence, although the glass insulators on the other two wires were in evidence.

In extra dry times the luminosity was somewhat greater and was frequently reported to be visible a half mile from the line, and was seen on many occasions as far as a mile from the line. The luminosity was of such a nature as to actually illuminate the surrounding objects, which made very perceptible shadows. A camera set at the power

Fig. 6. Pile Bridge Across Resurrection River Ready for Decking.

(All piles and caps cut from timber on right of way.)

house clearly showed ten spans of the single wire and showed three insulators on each of the poles in question. The telephone insulators on the first two or three poles were also evident in the photograph.

A telephone line ran about 6 ft. below the lower cross arm, and this line was charged so that it would flow one-half ampere through 100 ohms; i. e., it would light a 32-c. p. lamp to half its full brilliancy when connected directly between the combined telephone wires and ground. On open circuit, however, either of the telephone wires would discharge a spark 4 ins. long to ground terminal. In spite of this, however, the telephone was easily used, provided the operators were willing to chance the conditions of static charge. This was only tested by standing on insulated stools and not used for regular work.

No difficulty was experienced in throwing the current on the line during a fog or in any other weather.

One day after a Locke 2-11-23 was put on the line the outer piece was found to be split vertically through the top groove, but the inner piece held the voltage successfully except for a small amount of arcing, which caused the patrolman to shut the line down the next day and replace the insulator.

On April 18 a Knowles Imperial Porcelain 1-10-21 broke down and burned the top off the pole, making a short circuit and shutting down the line. This porcelain insulator was split down from the top a little to one side of the center in a manner similar to that shown by the Locke which had previously broken.

On April 29 a Muncie glass 1-9-16¾ broke down, shutting off the test. This insulator also had a vertical crack as though it were split down from the top and parted through the threads letting it drop free from the pin.

On May 21 a Knowles Imperial Porcelain 1-10-21 was broken off and carried away the top of the pole down as far as the second cross arm; the wires remaining clear until the arms fell, thus bringing the wires into direct contact.

During the entire test three wires on the other side of the poles, spaced with the same 40-in. distance, carried a 3-wire 15,000-volt line which was doing regular business and no difficulty whatever was experienced with it.

The conclusions arrived at by this test were that all is well while the insulators are new, or during any period when the insulators have no dust which can become mixed with fog or moisture by weather conditions.

No difference was experienced between wood and iron pins and in no case did the current short circuit from the wire to the pins over the insulators.

The results pointed out to the Pacific Light & Power Co. were that it would be well enough to use a large insulator. Their general knowledge of porcelain led them to adopt for the Kern River line a brown porcelain and the insulators were made of four pieces. The insulator was not unlike the usual 15-in. Locke insulator, known as No. 329. The only difference between the Kern River insulator and the Locke No. 329 is that the inner petticoats are spread somewhat in order to allow the company to apply what they consider to be the cure for all evils. This cure, which was to keep the insulators in as good condition as new for an indefinite length of time, was to have the lineman climb the pole once or twice a year, varying with the local conditions, and wipe out the inside of the insulator thoroughly with a cloth in order to remove the dust which might be collected there, at the same time inspecting the pole carefully. This, it is believed, will keep the insulators as good as new, and if they are so kept the company feels safe in assuming that 66,000 volts will cause them no difficulty whatever.

TENSILE IMPACT TESTS OF METALS.*

By W. Kendrick Hatt.†

INTRODUCTION.—In 1894 appeared Professor Merriman's interesting account‡ of the state of knowledge concerning the capacity of metals to withstand suddenly applied loads.

The author of the present paper was stimulated thereby to investigate the relative behavior of materials under impact and gradual loading. As the greater part of our present knowledge of the mechanical properties of materials is based on the results of tension tests under gradual loading, it seemed that the most direct comparison would result from tests in tensile impact. A vertical tensile impact testing machine was therefore constructed in 1897 and, since that year, experiments have been carried on in the Laboratory for Testing Materials of Purdue University, with the aid of a number of students whose patient and faithful services the writer here desires to acknowledge.‖ Other matters of a more or less practical interest have of late engaged the author's attention to the exclusion of these experiments. He is, however, loth to abandon this field without recording part of the results of the investigation, which, while incomplete, are nevertheless fairly definite in their indications. The results are interesting and may prove of use as a basis of correct thinking in relation to a matter which must have aroused the curiosity of many engineers who are concerned with the use of the materials of construction, and who have wondered if the static or gradual tests, now so universally used, give evidence of properties which fit a metal to withstand impact conditions.

The results described below lead to the conclusion that the ductility and shock-resisting capacity of metals of normal quality are not less under impact loading than those disclosed by the static test. If the tests had been continued to include materials that had proven defective in service, evidence would no doubt have been obtained to determine whether or not some form other than the static test is necessary to detect these defects. It should be noted that the speed of rupture of these tests corresponded to rupturing a tension bar that stretched 2 ins. in one one-hundredth of a second.

Recent work in Germany and France points to the conclusion that some form of impact test is found necessary to detect faults of structure that are not evidenced by the static test. It now appears that flexural impact test upon a nicked test bar is the best instrument for this purpose. The many mechanical difficulties inherent in the tensional impact test should, in the writer's view, prevent its use, especially in view of the fact that the flexural test is adequate to the purpose.*

DEFINITIONS.—The term "rupture-work" is taken to mean the work done in deforming a body to rupture. The area between the ordinary stress-strain diagram and the axis of deformations is a measure of this rupture-work. The latter term has often been replaced by the term "total resilience." Strictly speaking, the term "resilience" refers to the elastic energy stored up in a body at any stage of deformation.

"Unit rupture-work" is the rupture-work per unit of volume. Thus if a bar of steel ½-in in diameter and 8 ins. long is ruptured in tension as the result of a blow of a 512-lb. hammer falling from 5 ft., thereby bringing the hammer to rest, the total rupture-work will be 2560 ft.-lbs., and the unit rupture-work will be 1630 ft.-lbs. per cu. in. of metal.

MACHINES AND TESTS, TEMPORARY APPARATUS.—Two different machines, both vertical, have been used in these tests. For the tests on the specimens of steel wire 9 ft. long (Nos. 1 to 4 in Table 2) a temporary wooden machine was constructed as shown in principle in Fig. 1, and in construction in Fig. 2. This apparatus for producing longitudinal impact in tension was modelled after an apparatus used at Cornell University by Mr. Charles W. Comstock in impact tests of steel wire rope during 1898.† The principle therein employed, by which the hammer is hung on the specimen and impact is effected by sudden arrestment, during the fall, of an upper moving head, also fixed to the specimen, has been used in France in certain foundries.

In Fig. 2 may be seen the arrangements for hanging the hammer on a head which is fixed to the lower end of the test specimen by wedges. A similar head is fixed to the upper end of the test specimen. The writer improved this form of apparatus by supplying a rotating drum (D, Fig. 1), upon the surface of which a pencil, attached to the hammer, records a curve during the impact. The vertical ordinates to this curve, called a time-displacement curve, are the movements of the hammer; and the horizontal abscissæ are proportional to the speed of the surface of the drum. The latter speed is known from the record of a vibrating tuning fork, held against the drum. The velocity of the hammer may be computed from the drum record at any stage of descent.

In operation, the system (Fig. 1), consisting of ham-

*A paper presented before the American Society for Testing Materials, at Atlantic City, N. J., June, 1904.

†Professor of Applied Mechanics, Purdue University, Lafayette, Ind.

‡"The Resistance of Metals Under Impact"—Mansfield Merriman, Proceedings American Association for the Advancement of Science, Volume 43, 1894.

‖The experiments form the subject matter of the following undergraduate theses:

"Tests of Steel and Iron Rods Under Gradual and Impact Loading." L. V. Ludy and F. Nulsen—1898.

"A Study of the Behavior of Metals Under Impact." J. T. Nichols, J. W. Raub and T. G. Knauss—1899.

"Effect of Temperature on the Resistance of Iron and Steel to Impact." W. E. Jewell and G. S. Eaton—1900.

"A Study of the Resistance of Iron and Steel Under Impact." O. Klipsch and C. S. Johnson—1901.

"A Study of the Shock-Resisting Properties of Steel Castings." J. S. Tatman and N. J. Wheeler—1902.

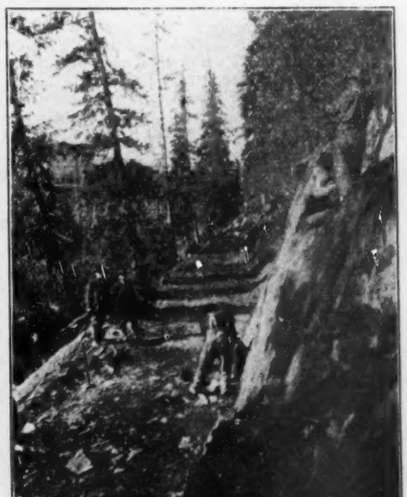


Fig. 8. Sidehill Work at Mile 11, June 1, 1904.

Fig. 9. Some Natives of Alaska.

*For an account of the state of knowledge relating to impact tests and for a bibliography of literature, reference is made to the following:

American Section, International Association for Testing Materials.—Bulletin No. 5, October, 1899. Report of Committee on Present State of Knowledge Concerning Impact Tests.—W. K. Hatt and Edgar Marburg.

Bibliography on Impact Tests and Impact Testing Machines.—Proceedings American Society for Testing Materials, Volume II., page 283.—W. K. Hatt and Edgar Marburg.

†Trans. Cornell Soc. C. E., Vol. 180.

mer, M, specimen B, and upper head C, is raised. Upon release, and subsequent descent, the upper head has impact upon a stop on bridge P, and the energy of the moving hammer thereby ruptures the test specimen. The pencil attached to the hammer is automatically released before the impact occurs, and the above mentioned record is thereby written on the surface of the drum.

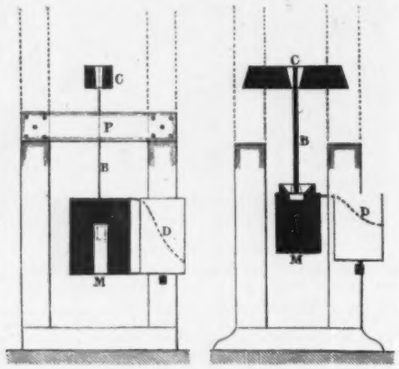
The energy remaining in the hammer after the rupture of the test bar is computed from a measurement of the velocity of the hammer at the time of rupture. This latter velocity is obtained from the slope of a tangent in the velocity-displacement curve.

PRINCIPLE OF ACTION.—As the discussion of the method of determining that part of the energy of the blow which is given up to the specimen will apply to the second machine described below, it is well to introduce the discussion here.

Fig. 3 is a representation of the record on the drum, and is thus explained:

With the upper head resting on the stop and the hammer hanging on the specimen, the pencil attached to the hammer is allowed to rest against the drum. The latter is rotated, and a datum line O O' is thereby traced. Any subsequent movement of the pencil below this line represents an elongation of the test specimen together with any yielding of the parts of the apparatus.

The system, consisting of hammer, specimen and upper head, is then hoisted and released. During the subsequent fall, the pencil attached to the hammer records on



Figs. 1 and 4. Diagrams of Temporary (Wooden) and Permanent (Iron) Machines for Testing Materials in Tensile Impact.

the revolving drum a parabola of free fall, A B. When the upper head has impact on the stop (P, Fig. 1) a retardation of the hammer is effected through the reaction of the test specimen. The curve B C is the record of the effect of the impact in retarding the hammer. The specimen is ruptured at the point C, and the record is then another parabola of free fall C E.

T T' is the record of the stylus fixed to the vibrating tuning fork, which is held against the drum during the impact.

The use of this drum record in computing the desired quantities depends upon the following considerations. Neglecting, for the present, losses of energy due to friction and yielding of the parts of the apparatus, let us suppose that the hammer falls from the point of release, a height, h + h', to some point D taken on the parabola of free fall subsequent to rupture. (The point C might have been selected, but the selection of the point D offers more favorable conditions for graphical work.) The entire quantity of work represented by the product of the

TABLE 1.—Comparison of Rupture-work as Obtained from I.—Load Elongation Diagram, and II.—End-Tangent Method. (Tensile Impact Tests of Annealed Steel Wire.)

Annealed Steel Wire.

Diameter, 0.246 in. Area of Section. .055 sq. in.

TEST A.

Gage length 108 ins. Drop 57 ins. Wt. of hammer 845 lbs.

No. of piece	Rupture work by diagram. Ft. lbs. per cu. in.	Rupture work by end tangent. Ft. lbs. per cu. in.
1	685	683
2	715	699
3	521	443
4	527	528
5	....	453
Average	612	561

Gage length 108 ins. Drop 70 ins. Wt. of hammer 845 lbs.

6	....	596
7	655	626
8	615	608
9	610	626
10	645	673
Average	631	626

TEST B.

Gage length 69 ins. Drop 38 ins. Wt. of hammer 845 lbs.

1	....	571
2	723	723
3	613	623
4	720	737
5	590	605
Average	665	633

weight of the hammer G into the distance h + h' is accounted for;

(1) By the work expended upon the resistance of the test specimen during elongation;

(2) By the energy remaining in the hammer at D, or, in the form of an equation,

$G(h + h') =$ Work on test bar $+ \frac{1}{2} M V_d^2$.

M = Mass of G.

V_d = Velocity at D.

The quantity $\frac{1}{2} M V_d^2$ may be considered as replaced by the quantity Gh_d where h_d equal the velocity head at D.

$G(h + h') =$ Work on test bar $+ Gh_d$

Or, Work on test bar $= G(h + h' - h_d)$

This sum, $h + h' - h_d$, may be considered as the effective distance through which the hammer falls in rupturing the test bar.

Evidently it is necessary to measure the velocity Vd in order to obtain the desired rupture-work of the test bar.

This velocity is obtained by drawing a tangent U U at D and measuring the relation of the vertical to the horizontal projections l' and l'', i. e., the relation of the velocity of the hammer to the velocity of the drum. The latter velocity is obtained from the tuning fork record. The duration of the impact will be the distance l'' expressed in seconds.

Thus to take an example from the test records:

Test No. 96.

Observations:
Tuning fork; 126 vibrations per second.
7 vibrations in 7.5 ins. of drum.
l' = 2.05 ins.; l'' = 3 ins.; h' = 1.6 ins.; h = 7 ft.

Calculations:
Velocity of drum = 11.25 ft. per sec.
Velocity of hammer at D = 11.25 × 2.05 ÷ 3 = 7.78 ft. per sec.
Velocity of head at D = 0.945 ft.
Rupture-work = 515 (7.133-0.945) = 3.186 ft.-lbs.
The volume of the bar under test was 1.63 cu. ins.; hence the unit rupture-work equals 3,186 ÷ 1.63 = 1,950 ft.-lbs. per cu. in.

This calculation neglects the friction of the guides, and other losses of energy due to the deformation of parts of the apparatus. As far as the friction of the guides during the greater part of the drop is concerned, this may be eliminated by drawing a tangent J J' at B. In all the tests made with the first temporary apparatus the velocity head at B, computed from tangent J J', was used instead of quantity h, thus eliminating the friction in the guides as far down as B. There were other losses of energy in this temporary machine, namely,

(a) At striking surfaces of upper head;
(b) Deflection of bridge;
(c) Slipping of wedges;
(d) Compression of heads of specimen;
(e) Pile action of the whole machine;
(f) General vibrations.

(a) The striking faces were metal and after a complete series of tests had been completed no noticeable damage had occurred at these surfaces. Therefore, these must have been little lost work in any one test.

(b) The total elongation of the specimen was generally 12 to 14 ins., and the greatest observed downward movement of the bridge was nearly ¼-in. This would cause an error of between one and two percent in rupture-work.

(c) and (d) The wedges were driven tight with a sledge hammer before impact occurred, thus taking up in advance any compression of the heads of the specimen and preventing slipping of the wedges.

(e) and (f) These cannot be measured. The writer assumes that the energy represented by vibrations is small.

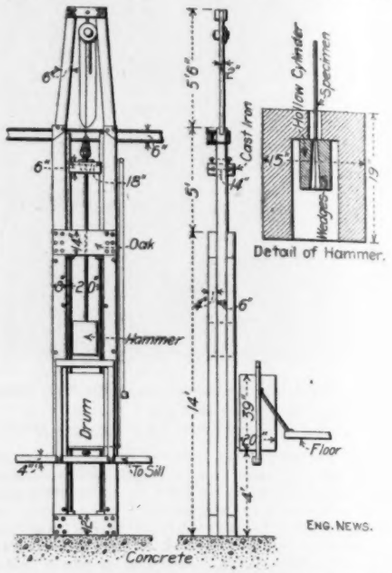


Fig. 2. Temporary Tensile-Impact Testing-Machine; Sketch at Right Shows Method of Fastening Hammer to Lower End of Test Specimen.

In the case of the long rods in which the elongation was as much as 14 ins. there was an opportunity to divide the curve B C into a number of parts. At each division point a tangent was drawn. The loss of velocity at the successive stages was computed from the observed change of slope of these tangents. Then the force necessary to account for the loss of velocity between the division points was computed by the principles of dynamics.

Let P = Retarding force in pounds.
p = Acceleration of hammer.
G = Weight of hammer.
T = Total tensional force in specimen.

$$G - T = \frac{Gp}{g}, \text{ or } T = G - \frac{Gp}{g}$$

In this manner the tension in the specimen was computed at various stages of elongation, and thus a "load-

TABLE 2.—Showing Comparative Tests Under Gradual and Impact Loading in Tension.

Gradual Loading Referred to Impact Loading as Unity.

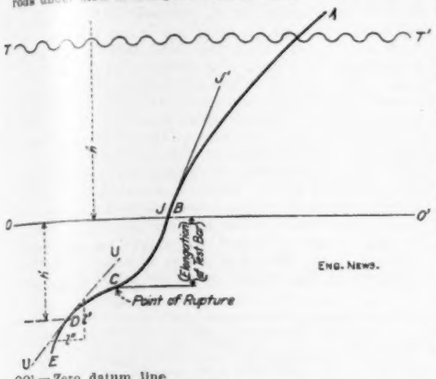
No. of test.	Material.	Diam. ins.	Gage length, ins.	Ultimate strength, lbs. per sq. in.	No. tested.	Elongation, %. Max.	Min.	Aver.	Rupture-work, ft.-lbs. per cu. in. Max.	Min.	Aver.	Kind of test.	Elongation, %. Max.	Min.	Aver.	Rupture-work, ft.-lbs. per cu. in. Max.	Min.	Aver.
1	Steel wire.....	0.16	48	11[illegible]	2	0.71	0.48	0.59	144	130	136	Gradual	0.71	1.14	0.84	0.67	0.78	0.74
		....	108		4	1.00	0.42	0.70	208	165	186	Impact	....	....	....	....	....	....
2	Steel wire, annealed	0.16	48		3	....	....	9.30	....	....	566	Gradual	....	....	0.99	....	....	1.04
		....	108		5	11.00	8.70	9.80	803	648	772	Impact	....	....	....	....	....	....
		....	48		4	8.00	6.00	7.00	554	534	544	Impact	....	....	....	....	....	....
3	Steel wire.....	0.30	72		5	15.80	12.80	14.10	865	690	742	Gradual	1.00	1.07	1.04	0.95	1.34	0.96
		....	108		5	15.60	12.00	13.00	903	515	765	Impact	....	....	....	....	....	....
4	Steel wire.....	0.26	108		5	4.64	4.62	4.63	349	347	348	Gradual	0.66	0.92	0.79	0.51	0.58	0.55
		....	108		5	7.00	4.00	5.10	699	443	556	Impact	....	....	....	....	....	....
		....	69		5	7.00	5.00	5.80	673	596	626	Impact	....	....	....	....	....	....
5	Nickel steel....	0.50	8		6	26.70	22.10	23.60	1,505	1,360	1,414	Gradual	0.90	1.06	0.98	0.67	1.13	0.77
		....	8		8	29.40	20.70	24.00	2,216	1,203	1,821	Impact	....	....	....	....	....	....
6	Soft steel.....	....	8	68,000	7	26.30	21.90	23.90	1,532	1,260	1,376	Gradual	0.83	0.94	0.83	1.05	1.04	1.01
		....	8	.....	15	31.60	23.10	27.00	1,460	1,208	1,358	Impact	....	....	....	....	....	....
7	Boiler plate steel	1.00	8	60,000	10	29.00	23.20	25.70	1,370	1,047	1,230	Gradual	0.78	0.79	0.74	0.67	0.64	0.62
		0.50	8	.....	4	36.90	31.30	34.40	2,035	1 690	1,855	Impact (2 blows)	....	....	....	....	....	....
8	Soft steel castings	0.50	2	62,000	5	29.50	26.50	28.20	1,666	1,342	1,450	Gradual	0.84	0.85	0.85	0.67	0.64	0.62
		....	....	.....	....	35.00	31.00	33.00	2,467	2,080	2,315	Impact	....	....	....	....	....	....
9	Hard st. castings	0.50	2	82,000	....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
10	Locom. tire steel	0.62	..	140,000	30	30.00	3.00	13.10	2,000	212	976	Gradual	0.81	1.00	0.91	0.35	0.23	0.33
		....	3	60,000	30	37.00	3.00	14.30	5,650	890	2,918	Impact	....	....	....	....	....	....
11	Norway iron.....	0.305	72	63,000	1	.....	.....	18.70	.....	.....	649	Gradual	....	....	1.03	....	....	0.98
		....	108	.....	3	19.40	16.70	18.10	727	550	636	Impact	0.81	0.97	0.90	0.69	0.79	0.77

Except where noted impact tests were made with one blow at atmospheric temperature on test bars of same length as used in gradual tests. The elongation is that after rupture (measured from broken ends). The rupture-work is that up to rupture.

elongation diagram" was constructed for the impact tests in tension. The evaluation of these diagrams should yield the same amount of rupture-work as that obtained by the process outlined above, namely, the process in which the velocity head after rupture was computed, called the "End Tangent Method." The accompanying Table 1 is a comparison of the unit rupture-work obtained by the two methods.

The experiments performed on this temporary apparatus include the following:

(1) A series of comparative tests of long steel and iron rods under slow and impact loading—1898.



OO' = Zero datum line.
AB = Free fall before rupture.
BC = Curve during Impact Elongation.
CE = Free fall after rupture.
UU' = Tangent to curve of free fall.
h = Height of fall.
h' = height from datum to point D, where tangent is drawn.

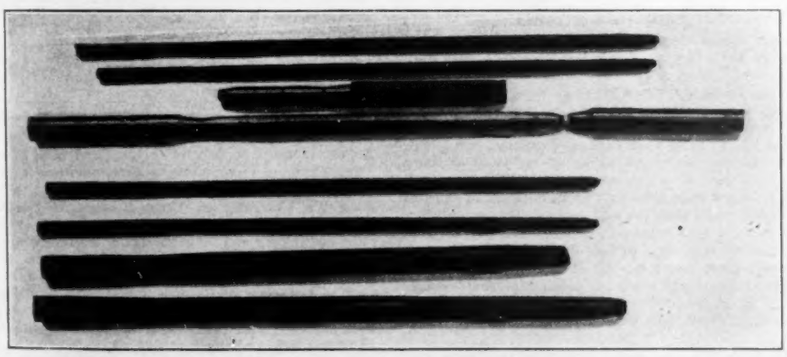
Fig. 3. Autographic Record and End-Tangent Method of Analysis.

(2) A series of tests of steel wire of different lengths, and the determination of the effect of temperature on tests of wrought iron—1899.

In these experiments the weight of the hammer varied from 845 to 1230 lbs., and the greatest height of fall was 6 ft.

In Table 2 the material under the reference numbers 1, 2, 3, 4 and 10 was tested on this temporary apparatus.

PERMANENT MACHINE.—The experiments performed on the temporary apparatus were successful enough to encourage the construction of a more permanent machine in metal. This machine has been described elsewhere in detail,* and will in this place only be described in general. A drawing of the machine was shown in Engineering News of Jan. 3, 1901. The permanent machine is similar to the temporary apparatus in principle (see a comparison in Figs. 1 and 4). Better adjustment for short specimens is brought about by dispensing with the bridge (P in Fig. 1) and constructing the upper moving head so as to span the distance between uprights as shown. This upper head or yoke is stopped directly by the lower sec-



The four upper specimens show the phenomenon of multiple necking in ordinary impact tests.
The four lower specimens were subjected to freezing along the center portion before and during fracture.

FIG. 6. IMPACT-TEST BARS SHOWING DOUBLE NECKING, AND EFFECT OF FREEZING CENTER PORTION.

tions of the columns. The details of attachment of the weight, recording apparatus, hoisting mechanism, etc., were as perfect as possible. The parts were designed to rupture, in one blow, a test specimen of soft steel ½-in. in diameter and 8 ins. between heads. Ordinarily the blow delivered is about 3,500 ft.-lbs. The machine rests on a brick foundation 5 ft. × 5 ft. × 6 ft. deep, laid in natural cement on undisturbed gravel foundation. The upper four courses are of paving-brick in Portland cement. A ¾-in. bearing bed of 1 to 1 Portland cement mortar is disposed between the bed-plate and the pier. The machine is also available for compressional tests.

An estimate was made of the energy lost in the various parts of the apparatus, thus: The parts of the apparatus that could be operated upon were loaded in a static testing machine and the consequent deformations observed. For example, the upper and lower yokes were placed in a statical testing machine with test bar and wedges attached and the deformation obtained under various central loads. From these observations the amount of energy absorbed by the upper or lower head at any given deflection could be computed. It only remained to observe the actual deflection which took place in these heads under the impact test to obtain a knowledge of the loss of energy corresponding to deflections of these heads in service. In this manner the main losses of energy could be estimated quite closely. The losses of energy at the striking surfaces cannot well be measured in any one test, but we may obtain some estimate of their amount by observing the cumulative effect at these striking surfaces after a long series of tests. The friction between the hammer and the guides can easily be measured either by statical methods or by finding the loss of velocity at different stages of the descent. The value of the acceleration of gravity measured by dropping the hammer and determining its velocity ran from 31.7 to 32.0 ft. per sec.

The author estimates the losses in this machine as follows, when tension bars of soft steel are tested:

At the upper and lower head, due to the deflection of the yokes 7 to 11 ft.-lbs. Friction of guides 8 ft.-lbs. At wedges, 0. For a total energy of blow 2,000 ft.-lbs., the energy not chargeable to specimen is less than 1%.

It may be said, with reference to loss of work due to the slipping of the wedges or compression of the heads of the specimen, that, before testing a specimen, the top wedges were driven tight with a sledge hammer and the bottom wedges pinched with a bar. Careful observations showed no slipping of the wedges due to the impact. What the losses of energy are at the striking surfaces of the upper head, and due to the vibration of the entire apparatus, etc., the writer cannot say. But since no permanent deformation seems to have resulted after some hundreds of tests, one must reason that the first of these is small.

In conclusion, the author believes that the uncertainties of the measurement of the amount of energy taken up by the specimen are small compared with the ordinary differences of quality as displayed under static tests of adjoining specimens in any rod or plate of even normal quality. The results obtained on this machine may, in the author's opinion, be relied upon for all practical purposes. The author does not share the feeling of those who would disregard the use of impact tests because of the small losses of energy that cannot be accounted for. This machine also serves as a laboratory apparatus for instructional purposes. Not only is the student impressed with the phenomena of rupture, but the observations and calculations bring home to him the mechanical relations of work, energy, etc.

In the experiments with this permanent machine, the weight of the hammer varied from 515 to 870 lbs., and the greatest height of fall was 6 ft. The work performed with the aid of this machine includes the following:

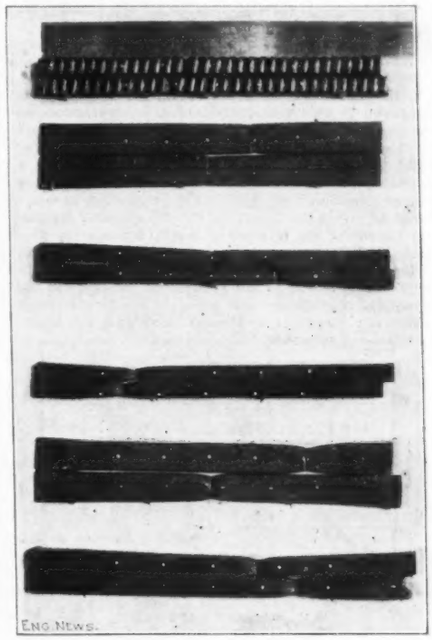
Tests of nickel steel and soft carbon steel in tension under gradual and impact loading. Effect of speed of application of a given amount of energy. Effect of temperature on the resilience of soft and nickel steel—1900.

Tests of steel castings in tension and flexure. Tests of cast iron in flexure. Tests of concrete under impact and compression—1901.

RESULTS.

COMPARISON OF IMPACT AND STATIC TESTS.—One of the first matters of interest is the relative behavior of material under impact and slow loading. It is also important to know how far the ordinary static test will yield information of the relative shock-resisting capacity of various materials. Table 2 is a tabulation of results of such comparative tests.

It appears from the results given that in the case of normal soft material, in bars of circular cross section, of lengths from 9 ft. to 8 ins., tested at ordinary temperatures, there is little difference in the total elongation and unit rupture-work whether the bar is ruptured in ten minutes or in from one to two one-hundredths of a second. The only marked point of difference in the static and impact tension test of soft materials of greater



The upper bar of each pair was broken by Gradual Loading.
The lower bar of each pair was broken by Impact Loading.

Fig. 5. Test-Bars Broken Under Gradual and Impact Loading in Tension.

lengths is that under impact any lack of homogenity of material is brought more into evidence. There is a tendency in non-homogeneous materials under impact to localization of the elongation of a test bar at various points in the gage length, so that the latter consists of nodes of ductility. After rupture the bar may have a number of incipient necks. In many cases two distinct necks are formed, as is shown in Fig. 6. The effect may be noticed in static tension tests, but not, in the author's experience, to the same degree as in impact tests. There is also a tendency, under impact, to localization of elongation near the wedges.

A marked difference between the results of gradual and impact tests may be noticed in case of short bars of steel castings and of tire-steel and coupons from boiler plate. There is a greater elongation and unit rupture-work under impact in the case of soft steel castings. But in the case of the harder steel, like tire-steel, the increase in unit rupture-work is disproportionate to the increase in elongation. This is partly due to the fact that there is considerable metal between the gage marks and the wedges in these steel castings, whereas the unit rupture-work is computed on the basis of the volume within gage marks.

The tests cover a range from soft steel castings to hard tire-steel, and a range of gage length from 2 ins. to 9 ft. All comparisons, however, made between static and impact tests of any one material are based on bars of the same gage length for the two kinds of tests, and represent tests in which the bars were broken in one blow except as where noted.

Table 2 shows the quality of the material and the average, as well as the maximum and minimum values observed. The comparison of the two kinds of tests given by Table 1 is graphically shown in Fig. 7, in which both axes are equal scales of the same kinds of quantity; if there were no effect of speed the plotted points would lie on the 45° line drawn on Fig. 7. Comparison of averages is plotted as a point, and also the comparison of the maximum values and the comparison of the minimum values. The elongation is always that obtained by measurements on the broken ends of the test bars. The rupture-work for the gradual tests was worked up from observations taken on specimens. In the case of long rods and very short rods this was done by direct measurement, and in the case of specimens of 8-in. gage length the rupture-work was obtained by the use of the Henning Pocket Recorder. An exception must be made in the case of tire steel.

*Engineering News, Jan. 3, 1901, or Nouveaux Appareils d'Essai—Congress International des Methodes d'Essai, Paris, 1900, Vol. 1, p. 507.

TABLE 3.—Soft Steel Castings, 2-in. Gage Length.

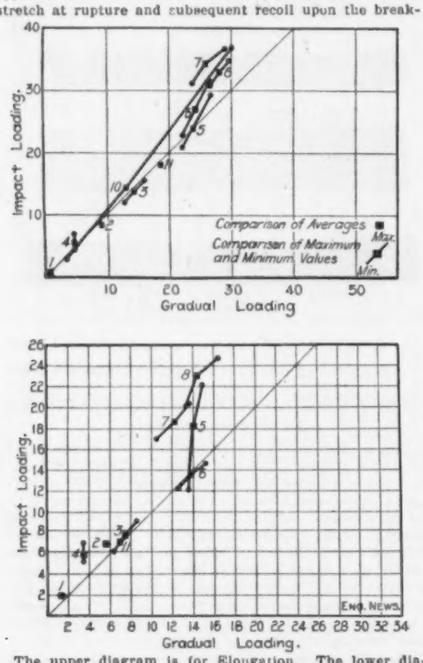
Kind of test.	Number tested.	Elongation Aver.	Elongation Max.	Elongation Min.	Rupture-work Aver.	Rupture-work Max.	Rupture-work Min.	Contraction Aver.	Contraction Max.	Contraction Min.
Gradual	5	28.2	29.5	26.5	1,450	1,666	1,542	46.0	54	40
Impact	5	33.3	35.0	31.0	2,315	2,440	2,080	44.2	40	40

TABLE 5.—Locomotive Tire Steel.

Number tested.	Elongation in per cent Max.	Elongation in per cent Min.	Elongation in per cent Aver.	Rupture-work in ft. lbs. per cu. in. Max.	Rupture-work in ft. lbs. per cu. in. Min.	Rupture-work in ft. lbs. per cu. in. Aver.	Kind of test.	Carbon.
6	10.0	3.0	7.16	.....	.....	.....	Gradual	Above 0.70
	13.0	3.0	5.90	3,920	904	2,403	Impact	
4	28.0	13.0	19.20	.....	.....	.....	Gradual	Above 0.70
	23.0	10.0	16.50	4,170	2,200	3,272	Impact	annealed.
4	21.0	10.0	26.20	.....	.....	.....	Gradual	0.60—0.69
	22.5	17.5	20.60	5 650	1,500	4,000	Impact	not annealed.

NOTE:—The Gradual results in table 5 are taken from mill reports.

It has been said in a previous paragraph that the elongation is always that obtained by measurements on the segments of the broken test bars. The rupture-work is, of course, the total rupture-work up to the point of rupture. In the case of these long rods there was a noticeable difference between the elongation of the rods as shown on the drum record and the elongation as determined by laying the broken ends of the specimen together and measuring the increase of length between the gage marks. This difference is too great to be accounted for by the supposition that the increased elongation as shown on the drum is due to the yielding of the parts of the apparatus. The difference is no doubt due to the elastic stretch at rupture and subsequent recoil upon the breaking of the bar. For instance, in No. 2 annealed wire (see No. 4, Table 2) the total elongation after rupture on broken test bar was, on the average, 5.46 ins., and the elongation shown on the drum just before rupture was 7.29 ins. This phenomenon was also noticed in the slow tension tests on the same material. Twenty-eight impact tests of annealed wire show an average difference between the elongation before and after rupture of 0.18 ins. per foot of length of bar. Six tension tests under gradual loading gave an average between these two measurements of elongation of .055 ins. per ft. The difference between the elongation before and after rupture is thus about ⅓ times as great in the case of impact tests as in the case under gradual loading. In the case of harder steel wire this difference between the two elongations was 0.12 ins. per ft., and was the same in gradual and impact tests.



The upper diagram is for Elongation. The lower diagram is for Unit Rupture-Work.

Fig. 7. Comparative Elongations and Unit Rupture-Works, Gradual and Impact Loading in Tension.

In the case of the tire-steel the rupture-work was computed from a knowledge of the results of static tests by substitution in a well-known formula which gives the approximate rupture-work. This formula is as follows:

$$\text{Ruture-work} = \frac{e}{300}\,(T' + {}^{3}T'').$$

Where e is the percent of elongation at rupture, T' is the elastic limit, and T'' is the ultimate strength.

The rupture-work of steel castings under gradual loading was also computed by the application of this formula to the results of gradual test made some time before in the laboratory by the writer, on bars of the same heat number. In order to check the application of this formula to steel castings of 2-in. gage length and ½-in. in diameter, a special series of tests was made on this material. The actual measurements during elongation were taken on 2-inch gage length and the stress-strain diagram therefrom resulting was evaluated in order to obtain the rupture-work. The actual measurement of rupture-work was compared with the results from the application of the formula given above. This showed that the formula applies with a small limit of error.

Information is given in Tables 3, 4, and 5 more in detail concerning the properties of steel castings and locomotive tire steel under impact. Table 3 shows a special comparison of five soft annealed steel castings of a fine silky fiber under impact and gradual loading, when broken with one blow of a 580-lb. hammer, falling through an effective distance of about 1.45 ft. Table 4 gives the average properties of steel castings of different character as shown by the appearance of fracture. Table 5 gives detail information of the same kind with respect to locomotive tire steel.

The material in Table 2 is described as follows: The steel wire specimens were cut from one length of wire which was either annealed or not annealed as indicated. This wire was supplied by the Trenton Iron Company. The nickel-steel was sent from the Carnegie Steel Company and was from one heat and rolling. Analysis as follows:

	per cent.
Carbon	0.21
Phos.	0.013
Mang.	0.60
Sulph.	0.026
Nickel	3.15

The soft steel was purchased on the market in the form of rods of 16-ft. length. The boiler plate specimens were coupons from plates rolled for a single order to be used in locomotive boilers. Steel castings were machined coupons which were cast on the rims of locomotive driving wheel centers, and were generally annealed. They were supplied by the Peru Steel Casting Company, Peru, Ind., from heats selected by the writer on the basis of static tests. The locomotive tire steel was sent from the mill of the Standard Steel Works. The static tests on the tire steel were derived from the mill reports. The comparison quoted includes both annealed and unannealed bars. This comparison of tests on locomotive tire steel is somewhat unsatisfactory in detail for the reason that some of the bars are annealed and some not annealed. The average, however, is significant.

TABLE 6.—Variation of Elongation and Rupture-work with Gage-Length.

Tensile Impact Tests.

Gage length.	Machine steel. Elongation.	Machine steel. Rupture-work.	Soft steel. Elongation.	Soft steel. Rupture-work.	Annealed steel wire. Elongation.	Annealed steel wire. Rupture-work.
3-in.	100	100	..	..	..	..
4.5-in.	98	89	..	..	..	..
5-in.	83	82	..	..	..	..
5.5 in.	77	66	..	..	..	..
6-in.	92	..	..	..	..	..
6.5-in.	104	93	..	..	..	..
7-in.	89	79	..	..	..	..
8-in.	84	78	..	..	..	..
11-in.	..	..	71	60	..	..
12-in.	..	..	..	..	23	50
14.5-in.	73	60	..	..	..	..
18-in.	..	..	64	56	..	..
19-in.	..	..	70	60	..	..
23-in.	..	..	57	52	..	..
25-in.	..	..	61	56	..	..
25.5-in.	..	..	..	..	18.5	36
26-in.	..	..	..	..	18.5	32
108-in.	..	..	..	..	15	25

Results are based on the value of 3-in. gage length as 100.

THE EFFECT OF VARIOUS FACTORS ON THE RESULTS OF IMPACT TESTS.

GAGE LENGTH.—In static tests both elongation and rupture-work increase with diminished gage length. The effect of change of gage length upon the results of impact tests was investigated in order that results might be reduced to a common basis when bars of varying gage length have been tested. Three different kinds of material were used. Series A consisted of a number of bars machined, varying in gage length from 2½ ins. to 14½ ins. The specimens were all broken with one blow of a hammer weighing 515 lbs. These specimens were machined from rods to a section ⅝-in. in diameter.

Series B consisted of rough soft steel bars, 5-16-in. diameter varying in gage length from 11 ins. to 25 ins. Series C consisted of specimens of annealed steel wire varying in length from 12 to 108 ins. The results are shown in Table 6.

The contraction of area seemed not noticeably affected by the gage length, but it is very evident that the unit rupture-work and percent of elongation increase as the gage length decreases. This effect is due (1) mainly to the work of deforming the bar at the neck, which work bears a greater proportion to the total work in the case of short gage length and so increases the rupture-work per cubic inch; and (2) partly due to the fact that the work done in stretching the metal between the gage marks and the wedges is a greater proportion of the total. The increase in percent of elongation and unit rupture-work may not be expected to increase indefinitely, because when the reduced section becomes a "nick" both of these diminish. There must be then a particular gage length for maximum value of these quantities.

TABLE 4.—Steel Castings of Various Kinds.

Character of fracture.	Kind of test.	Elongation.	Rupture-work.	Contraction.
Silky	Gradual	26.0	1,338	50
	Impact	30.0	2,100	50
Flaky,.....	Gradual	21.6	981	45
	Impact	19.0	900	45
Fine bright	Gradual	17.4	744	38
	Impact	22.6	2,811	50

NUMBER OF BLOWS.—It was the intention in all the writer's tests to produce rupture in one blow. Sometimes, however, unknown material came into the laboratory, the impact resistance of which was not known in advance, and the bar had to be broken sometimes by two, sometimes by three, blows. It seemed necessary, therefore, to determine the relative properties exhibited by material when tested with one or two or three blows, as the case might be, applied a few minutes apart, in order that the result might be reduced to the standard of one blow.

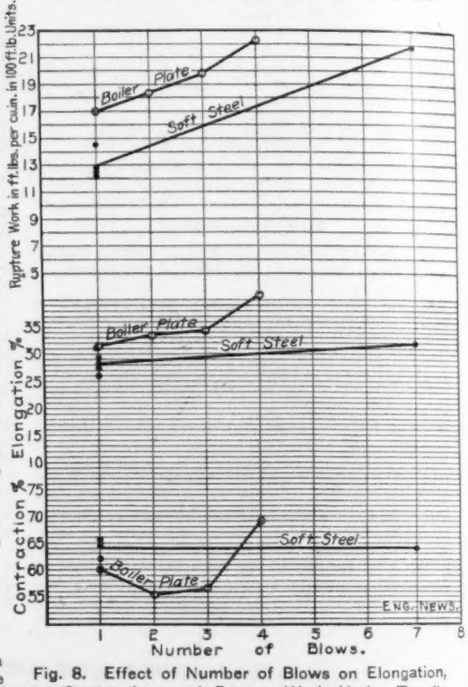


Fig. 8. Effect of Number of Blows on Elongation, Contraction, and Rupture-Work Under Tensile Impact.

To determine these relations three series of bars were used. Series A consisted of five specimens of boiler plate (No. 7 in Table 2). Series B consisted of specimens of rough bars of soft steel. Four specimens were broken with one blow, and one specimen broken with seven blows. Series C consisted of steel castings 2 ins. in gage length and ½ in. in diameter, broken with either one blow or a number of six-inch blows. The results are as shown in Table 7, and Fig. 8. The ductility and rupture-work increase with the number of blows in spite of the hardening effect of the first and succeeding blows. In a test of a steel rod under twelve blows from 1-ft. height, the hardening was evident in its effect on the recoil or rebound of the hammer. The recoil of the hammer from

TABLE 7.—Showing Comparative Effect of One Blow and a Number of Blows on Elongation and Rupture-Work.

Tensile Impact Tests.

No. of blows.	Boiler Plate. Elongation.	Boiler Plate. Rupture work.	Soft steel. Elongation.	Soft steel. Rupture work.	Soft steel castings. Elongation.	Soft steel castings. Rupture work.	Hard steel castings. Elongation.	Hard steel castings. Rupture work.
1	100	100	100	100	100	100	100	100
2	106	....	....	....	....	....	....	....
3	110	....	....	....	....	....	....	....
4	132	....	....	....	....	....	169	450
5	....	....	....	....	102	130	....	....
6	....	....	....	....	....	....	....	....
7	....	....	115	....	....	....	....	....
8	....	....	....	....	....	....	....	....

Values quoted are on basis of 100 for value at one blow.

TABLE 8.—Effect of Speed of Delivery of Energy.

Series A—Annealed Steel Wire 0.26 inch Diameter.

No.	Length.	Number tested.	Wt. of hammer, lbs.	Height of drop, ins.	Rupture-work. Max.	Rupture-work. Min.	Rupture-work. Aver.	Elongation. Max.	Elongation. Min.	Elongation. Aver.
1	108	5	845	57	683	443	609	6.95	4.30	5.06
2	108	5	845	70	673	596	626	6.90	4.60	5.71
3	108	5	1,230	36	680	497	487	5.40	4.00	4.76

Series B—Soft Steel, ½ inch Diameter x 8 inches Long.

5	8	5	515	60.5	1,471	1,237	1,365	30.00	25.00	26.2
6	8	8	810	36.5	1,502	1,215	1,317	24.00	32.00	27.3
7	8	8	Gradual test.		.....	.....	1,370	.....	.....	24.0

TABLE 9.—Variation of Elongation and Rupture-work with Temperature.

Tensile Impact Tests.

Temp. Deg. F.	Soft steel. Elongation.	Soft steel. Rupture-work.	Nickel steel. Elongation.	Nickel steel. Rupture-work.	Norway Iron. Elongation.	Norway Iron. Rupture-work.
—100	33	41	65	81	....	....
0	73	74	79	96	....	....
32	100	102	....	....	26	48
40	....	....	113	102	....	....
70	100	100	100	100	....	....
80	....	....	....	....	100	100
212	104	100	100	82	165	138
400	94	80	87	61	....	....

Values quoted are on basis of 100 for value at 70° F.

TABLE 10.—Effect of Temperature on Elongation at Fractured Inch.

Tensile Impact Tests.

Temperature.	Inches elongation of fractured Inch. Nickel steel.	No. of tests.	Soft steel.	No. of tests.
—100	0.40	3	0.46	1
0	0.44	6	0.48	3
80	0.49	8	0.49	8
212	0.53	1	0.52	2
400	0.54	2	0.45	1

the first blow was 0.3 in.; from the sixth, 0.5 in.; from the tenth, 0.8 in. The rupture-work quoted in the tables does not include the minor blows due to recoil following the parent blow.

VELOCITY OF DELIVERY OF A GIVEN AMOUNT OF ENERGY.—It seemed necessary to determine if, within the ordinary limits of impact tests on the machine, the speed of delivery of a given amount of energy had an important effect on the results. That is to say, are the results the same whether the energy is delivered through the medium of a heavy weight falling through a small distance or whether through the medium of a lighter weight falling through a greater distance? Of course in extreme limits there would be noticeable differences of effect. The writer's experiments directed to this question are shown in Table 8. The conclusion is that the results are not sensitive to the ordinary changes of velocity of application to be expected in any one impact machine.

TEMPERATURE OF TEST BAR.—To determine the law controlling the variation of resistance in change of temperature under impact, the writer has used specimens of soft steel and nickel-steel (Nos. 5 and 6 in Table 2). The specimens were broken with one blow each.

The method of producing the desired temperature in the test bar is as follows: To obtain a temperature of 32° F. the test bar, together with the wedges, was allowed to lie in cracked ice for 30 minutes before test. The bar was then surrounded with an asbestos wrapped tin cylinder 2 ins. in diameter and 9 ins. long, which held cracked ice, and then rapidly fixed in the machine. With the tin cylinder still on the bar, the system was hoisted, released and impact then occurred. The time from the removal of the bar and the wedges from the cracked ice until the time of impact is estimated at about 8 minutes. Of course during this period the guides and yokes of the apparatus were delivering heat to the wedges and test bar, and thereby producing a gradient of temperature between the center of the bar and the upper and lower ends. The effect of this gradient is to tend to bring about rupture at the ends of the specimen rather than at the center. The temperature of 0° F. was effected, in the same manner as above described, by a mixture of salt and ice. The temperature of —100° F. (estimated) was secured by a freezing mixture of carbonic-acid snow and ether, kept in the form of a slush. In all cases the bars were packed in the mixture before test and the cylinder containing the freezing mixture was placed around the bar during the entire period of the test. The temperature of the test bar is assumed to be that of the mixture. The temperatures of 212° F. and 400° F were secured by filling the tin cylinder with a fluid, heating the fluid by a gas flame while the specimen was in the machine ready to be dropped. Boiling water was used for 212° F., and boiling linseed oil for 400° F. All the temperatures were measured (except that of the slush of carbonic-acid snow and ether) just before the drop occurred while the bar was in the machine surrounded by the freezing mixture.

The results of this investigation are shown in Table 9 and Figs. 9 to 11.

It appears from these results that in the case of ordinary soft steel there is a diminution of elongation and unit rupture-work as the temperature goes below 32° F. or above 212° F. In the case of nickel steel this diminution is apparent when the temperature goes below 32° or above 70°. The contraction seems not to be much affected by change of temperature. Even in case of the bars cooled to —100° F. the fracture was so hot that the broken end of the bar felt uncomfortably hot to the flesh.

A study of the distribution of the elongation in the ruptured bars shows that the bars which were heated above atmospheric temperature have a tendency to break near the center, whereas bars broken at low temperature break near the ends. At —100° the elongation of the bars of soft steel was not more than 1% at any inch which was at least 2 ins. from the point of fracture. In many cases the part of the bar near the center was not stretched to a measurable degree. The nickel steel

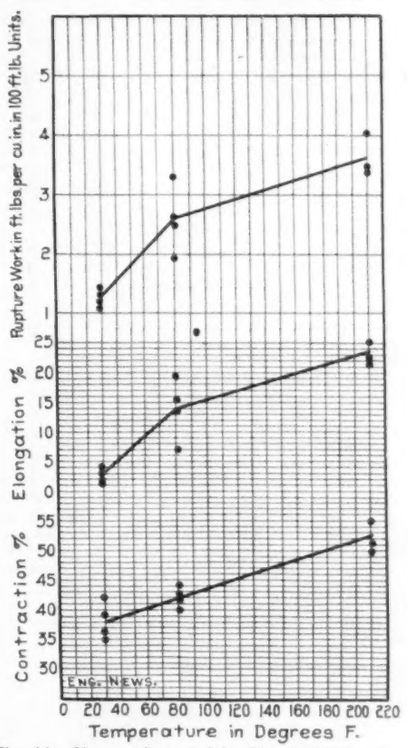


Fig. 11. Norway Iron, 1-4 in. Dia. by 60 ins. long.

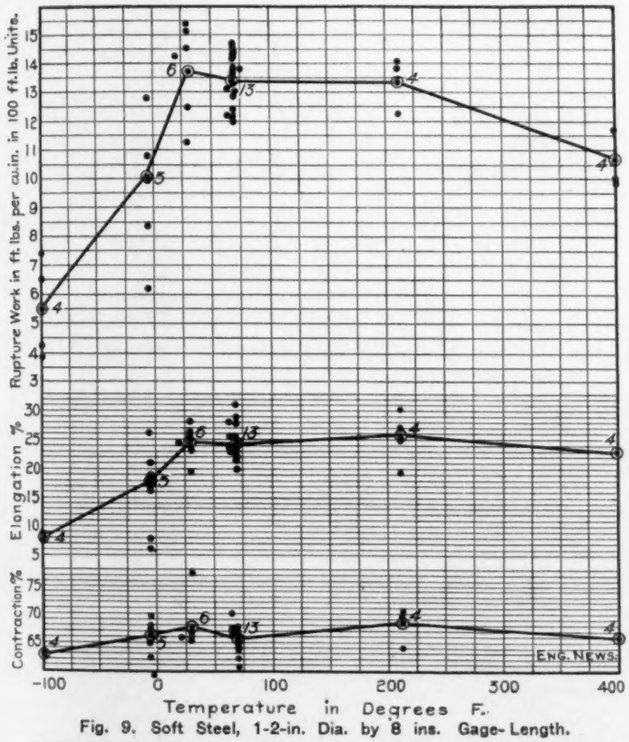


Fig. 9. Soft Steel, 1-2-in. Dia. by 8 ins. Gage-Length.

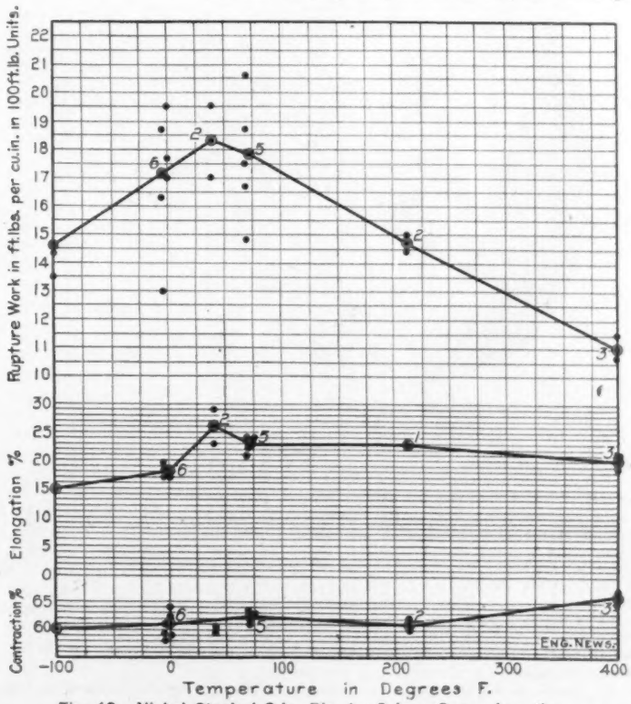


Fig. 10. Nickel Steel, 1-2-in. Dia. by 8 ins. Gage-Length.

FIGS. 9, 10 AND 11. EFFECT OF TEMPERATURE ON ELONGATION, CONTRACTION AND RUPTURE-WORK FOR VARIOUS KINDS OF STEEL, UNDER TENSILE IMPACT.

TABLE 11.—Effect of Shape and Surface of Boiler-plate on Elongation and Rupture-Work.
Original Bar: Sides as in Original Plate, Milled Edges, 8-in. gage length, 1 x 0.5 in. Cross Section.

Modification of specimen.	No. tested.	—Elongation, per cent.— Max.	Min.	Aver.	Rupture-work in ft.-lbs per cu. in. Max.	Min.	Aver.	No. of blows.	Kind of test.
Original	19	29.0	23.7	25.7	1,370	1,047	1,230	..	Gradual
	4	36.9	31.3	34.4	2,035	1,690	1,855	2	Impact
No fillets	1	....	....	27.0	.....	.....	1,260	..	Gradual
	2	28.0	27.0	27.5	2,791	2,391	2,591	2	Impact
Nicked cross section	1	....	....	8.5	.....	.....	180	..	Gradual
	2	0.12	0.11	0.11	385	374	379	1	Impact
Polished all four faces	0	....	....	....	.....	.....	....	..	Gradual
	2	29.0	29.7	28.2	2,312	2,169	2,240	2	Impact
Rasped, four faces	0	....	....	....	.....	.....	.....	..	Gradual
	3	27.0	16.0	21.6	2,226	1,106	1,682	1-3	Impact
Edges rounded to semi-circle	1	....	....	26.5	.....	.....	1,116	..	Gradual
	2	34.0	27.0	30.5	2,052	2,019	2,035	2	Impact
Turned and polished, dia. 0.56 in. Area = .245 sq. in.	0	....	....	....	.....	.....	.....	..	Gradual
	2	20.0	18.7	19.3	1,021	766	893	1	Impact
Milled and polished, sec. 0.35 x 0.70. Area = .245 sq. in.	0	....	....	....	.....	.....	.....	..	Gradual
	3	25.0	23.5	24.1	1,571	1,491	1,532	1	Impact
Milled to square cross-section .063 ins. on side	0	....	....	....	.....	.....	.....	..	Gradual
	2	31.0	27.8	29.4	.....	.....	1,990	2	Impact

showed a more uniform elongation. This, in the writer's view, is due to the fact that under impact a bar tends to break at the warmer portions. To investigate this matter more thoroughly a few bars were purposely given a gradient of temperature. Thus the center of the bar was surrounded with the freezing mixture and the ends of the bar were heated by a gas flame; the frozen portion of the bar retained its original diameter and was not stretched by the blow, whereas the warm ends were drawn down to necks. Two bars were frozen at the upper ends and the lower ends were exposed; these broke at the lower ends. The frozen portions had been able to transfer the strain energy quickly throughout their length to the warmer portions where this energy was absorbed in deforming the material.* Fig. 6 shows some of the test-bars.

This result might have been expected from a knowledge of a fact which has been determined experimentally, namely, that the effect of low temperature is to raise the

Fig. 1. View of Phillips Pressed Steel Pulley.

elastic limit of metals. Now it is another well known fact that when two materials are placed in conjunction, one of which is rigid and the other a cushion, that the more rigid material will quickly transfer the strain-energy to the cushion where it is absorbed. For instance, the function of the hardened face of an armor plate is to transfer the energy of the projectile quickly throughout the metal of the face to the soft back where it is absorbed without breaking the plate. The effect noticed in the author's experiments may thus be explained by the assumption that the colder portions of the bar were transformed, in effect, to a metal of a higher elastic limit, which had the property of transferring the strain-energy to the warmer portions of the bar. This frozen portion, therefore, did not suffer any elongation. The total rupture-work was of course less than that obtained from a bar which was entirely warm, for the reason that there was less metal undergoing deformation. The energy has to be absorbed somewhere in the bar, and where the entire bar is cold there is less capacity for absorbing shock up to rupture. However, a cold bar should withstand ordinary shocks of service within the elastic limit with a greater factor of safety since the effect of low temperature is to raise the elastic limit.

It must be recognized that the laws shown in Figs. 9 to 11 should be modified by magnifying the variations there shown, for the reason that the temperature of the test bars under the freezing mixtures was not uniform, but that the full weakness due to the temperature was somewhat reduced by the higher temperature of the ends of the bars. The exact reductions the author is unable to estimate.

To meet the criticism that the decrease in elongation at low temperatures is mainly due to the fact that the bars did break at the end and not to the effect of low temperatures, the writer examined the relative elongation of bars broken at normal temperatures in the middle third of gage length, and at the ends. Thus, the average per cent of elongation when broken at the center was 29.6; and when broken at the ends 26.4%. The elongation at —100° F was 8%. About one-half of the number of bars broken at 70° ruptured near end gage mark. Thus, the decrease in elongation in Fig. 9 is evidently too large to be accounted for by the location of the fracture. Table 10 gives the elongation at fractured inch of gage length as affected by temperature.

CONDITION OF SURFACE AND SHAPE OF CROSS SECTION.—The author had planned a series of tests to determine the relative properties of bars, some of which were to be polished, others rough, others rasped, etc. Another series was to include bars of rectangular square and circular cross section, all of the same area. It seems reasonable to suppose that the phenomenon of the flow of metals under high rates of distortion must be influenced very largely by the shape of the cross section and the condition of the surface.

The experiments were not completed, but a series of boiler-plate test bars was tested with the results shown in Table 11. The bars were selected from a large number of specimens so as to be as nearly as possible of the same quality as shown by the test reports from the mill. The results of these tests are not, in the author's opinion, conclusive, but they show indications. For instance, polishing the bars did not cause any large increase in ductility over a bar with ordinary mill surface. When a bar, however, is taken rough from the first cut in the milling machine, the strength is less than a bar that has been given a finishing cut. The nicked bars, of course, had but very little strength.

*However even in impact tests at normal temperatures there is a tendency to elongate and rupture near the ends of the bars, rather than at the center.

A NEW DESIGN OF PRESSED-STEEL PULLEY.

Pressed steel sheet or plate has found successful application to such a wide variety of uses that a new or modified application awakens little attention or comment. When something new in pressed-steel is announced, it is dismissed with the comment, "That means nothing but a certain amount of special machinery. Provided he can do enough business to pay for this investment, the problem is simple enough." Such passing comment wholly neglects the fact that much ingenuity and careful design are called into action in making possible the application of pressed-steel to the case in hand. Of course, the material is cheap, is available in uniform, excellent quality, and presses for working it into a great range of forms are to be had. But, sheet-steel is of small thickness, and is no thicker at the most strongly

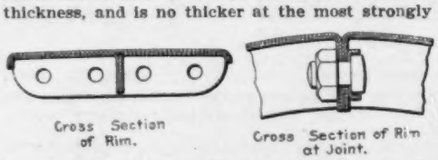


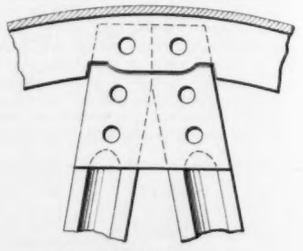
Fig. 2. Sections of Rim, Showing Method of Forming Rim and of Joining Halves.

stressed section of a piece than at other sections; proper shaping of the piece must be secured, then, to compensate for the differences in stress. Moreover, the different pieces that make up the desired whole must be joined by means that are at once cheap and permanently secure. The advantages of lightness, elasticity and strength over the competing material, cast-iron for example, must not be lost by less economical distribution of material, or by greater clumsiness and cost of connections. These and other considerations combine to make the use of pressed-steel design dependent on careful design fully as much as, or more than, merely the cost of the necessary plant.

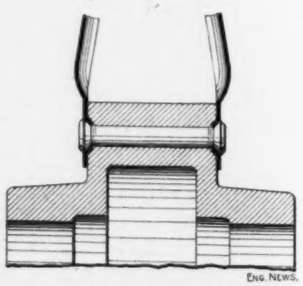
An illustration of the extent to which design enters into the production of pressed-steel articles is offered by the pressed-steel pulley, which is described in brief below. The description is based on information supplied us by the manufacturers, the Philips Pressed Steel Pulley Works, of 4th St. and Glenwood Ave., Philadelphia, Pa.

The pulley, shown in general view in Fig. 1, is designed as a split pulley, and is closely similar to the widely used wood split-pulleys; like these, it is adaptable to shafts of different diameters by using removable bushings of suitable sizes. The pulley is made of pressed-steel, with the exception of the hub, which is of cast-iron. This hub is made in two halves joined by compression bolts, which enable the pulley to be fastened on the shaft by compression alone. Keys or set-screws may be used in addition where thought desirable.

The rim of the pulley consists of a double thickness of steel sheet, being made from a single wide sheet, folded and flanged as indicated in Fig. 2. The double central rib, where the two edges of the original sheet are brought together, provides for the attachment of the arms. The two



Connection of Arms to Rim.



Connection of Arms to Hub.

Fig. 3. Connections of Arms to Rim and Hub.

rim-sections, each a semicircle, are joined by bolts passing through flanges formed by bending down from the metal of the rim. The bolts which make the joint are locked by a small channel-shaped strip under their heads, so that they are held against turning when the nuts are tightened up.

Some important points of detail design are exhibited in the construction of the arms, and their connection to rim and hub. The arms are arranged in sets of four, having separate points of attachment to the hub, but converging to a common connection at the rim. This gives a structure braced against the circumferential tractive load and against lateral pressures. Each arm is pressed into approximately semi-circular cross-section, giving it the necessary compressive strength.

At the rim, where the four arms of a group converge, they are riveted to a single central gusset-plate (Fig. 3), which has tenons projecting into notches cut into the central rib of the rim, above mentioned. The arms themselves pass on the outside of the rib, and are riveted through the rib by two rivets. The arrangement of the gusset-plate provides for transmitting the circumferential load through the tenoned connection, so that the rivets in the rim have merely to hold the parts firmly together.

A similar principle is carried out in the connection of the arms to the limb. This connection also is shown in Fig. 3, the plane of section being parallel to the shaft. Two opposite arms of a group are held by a single pin passing through a projecting lug of the hub, riveted over on both ends. But in order to relieve this pin of shear, the metal of the arms is pressed into countersunk recesses in the hub, under the heads of the pin. The construction is clearly shown in the drawing.

The arms and rim of these pulleys may evidently be made of metal of any desired thickness (and therefore strength) as may be selected for any individual case. A pulley operating under heavy load and with very tight belt would naturally require to be stronger than one intended for light service.

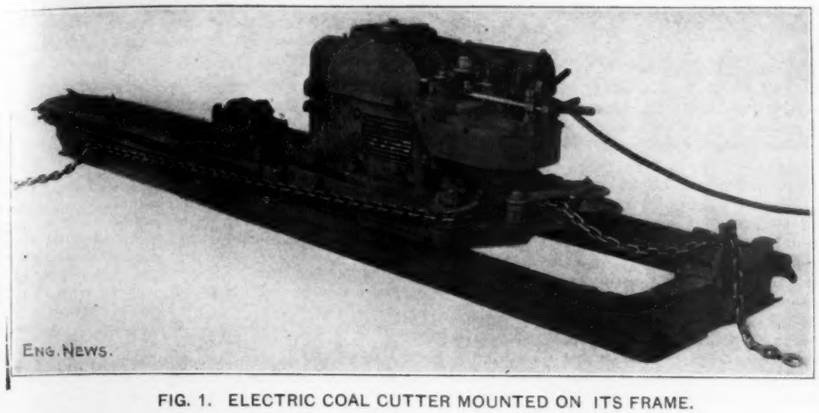


FIG. 1. ELECTRIC COAL CUTTER MOUNTED ON ITS FRAME.

SOME RECORDS OF WORK DONE BY AN ELECTRIC COAL CUTTING MACHINE.

The accompanying illustrations, Figs. 1 and 2, show an electric chain coal undercutting machine which differs from others previously on the market. The essential feature is that after the sumping cut is made, the machine is converted into a longwall cutter and makes a continuous undercut the whole width of the face, traveling by its own power on a steel shoe without the use of rails or wheels. The result of this method of longwall cutting is a minimum of time lost in setting up and shifting the machine. The following records serve to show what the machine is capable of doing when properly handled:

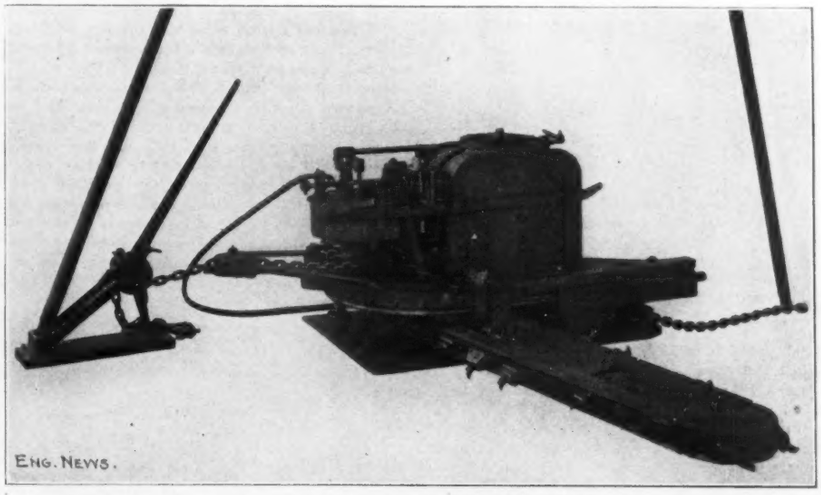


FIG. 2. ELECTRIC COAL CUTTER REMOVED FROM ITS FRAME.

Reference No.	Mine.	Length of long wall face.	Time cutting	Depth of cut.	Height of vein.	% lump, hand mining.	% of lump after machine.	Electrical HP.	Feed per min.	Feed chain pull.	Pitch of vein.
		ft.	hr. min.	ft.	ft. in.	%	%	HP.	in.	lbs.	deg.
1	Bolen-Darnall C. Co., Craig, I. T.	311	6 30	5	3 6	45	90	21	21	5,200	23
2	Bolen-Darnell C. Co., Craig, I. T.	385	8 00	5	3 6	45	90	21	21	5,200	23
		115	....	..	....	...	..	..	..	4,100	..
		500									
3	Colorado F. & I. Co., Fremont, Colo.	306	6 30	5	4 0	55	80	26	24	4,800	8
4	Victor Fuel Co., Hastings, Colo.	160	1 20	6.6	8 0	40	70	17.5	27	5,400	7
5	Northern C. Co., Aguilar, Colo.	285	2 15	5	4 0	60	80	18	36	5,300	6

Remarks:
1. Cutting up across pitch in hard fire clay; cut in bottom.
2. Cutting down across pitch in dark fire clay; cut in bottom.
3. Cutting down across pitch in medium hard fire clay; cut in bottom.
4. Soft cutting.
5. Soft cutting.

Monarch Mine, Marshall, Colo. (20 ft. rooms and 8 ft. entries) 11 places in nine hours, 37 places in four days, 85 places in 12 days. Curtis Mine, Colorado Springs, Colo., one machine, coal 9 ft. high, 7,000 tons in 25 shifts work. Vesta Coal Co., California, Pa., eleven 30-ft. rooms in one 7½ hour shift.

The accompanying records, made in the Mine of the Vesta Coal Co., Allenport, Pa., show the time required for the operation of the machine in its various processes. The coal was medium soft; depth of cut, 6 ft.; height of coal, 8 ft.; average horse power consumed by machine, about 9¼ HP.; feed, 21 ins. per min.; pull on feed chain, about 3,200 lbs.

Based upon such records as the foregoing, the manufacturers make the following claims for this type of coal cutter: (1) It will mine 50 to 150 per cent. more coal than other coal cutters.

(2) Less power is required per ton of coal mined, 20 HP. being required while cutting, and 4 HP. while loading or unloading and moving about the mine.

(3) The kerf is higher than that of any other chain machine, being 5½ to 6 ins., but consists of less slack and more nut and pea coal.

(4) As the machine makes a continuous cut, it leaves no stumps or sprags, as the breast machine does, to interfere with the proper blasting of the coal, thus requiring less powder and increasing the percentage of lump coal.

(5) The machine cuts very close to the bottom, leaving practically no bottom to be taken up. By means of a taper skid or by raising or depressing the feed chain, quite sharp irregularities may be followed. The machine will follow ordinary rolling bottom without assistance.

(6) Timbers may be set about 5 ft. from the working face, being about one-half the distance required for the breast machine, insuring the safety of both machine men and loaders. Where draw slate comes with the coal it is necessary to move it only one-half the distance required for the breast machines; or in many cases it need not be removed at all.

(7) Owing to the continuous nature of the cut and the method of setting the jacks, there is no vibration to affect a weak roof.

(8) The machine propels itself by power to any desired position in loading or unloading and in moving about the mine.

(9) The machine will operate on any pitch up to 30°, when rooms are driven across the pitch; it will work straight up the pitch, providing this does not exceed 12°. This advantage in pitching veins is noteworthy and has solved the problem in work of this character. In the pitching veins of Indian Territory, where three men are required to handle a breast machine, only two men are required to work this machine.

This coal cutting machine is made by the Sullivan Machinery Co. of Chicago, Ill.

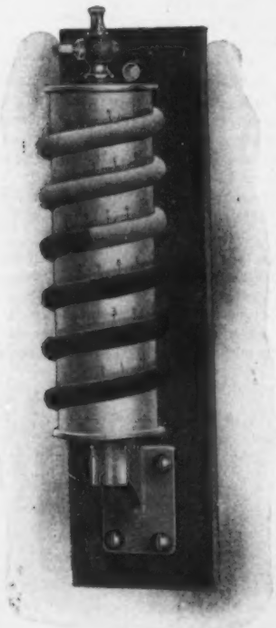
Records Made by Electric Coal Cutter in Vesta Coal Co.'s Mine, Allenport, Pa.

Room No.	Condition of room.	Time unloading and placing machine.	Time cutting sumping cut.	Time detaching pan and setting jacks.	Time cutting face of room.	Time loading.	Time moving to next room.	Distance moved.	Total time of cutting room.	Width of room.	Net tons.
		min.	min.	min.	min.	min.	min.	ft.	min.	ft.	tons
18	Bottom good	5	4	6	14	6	8	....	43	31	55
16	Bottom good	5	4.5	5	16	5.5	6	470	42	28	49
19	Bottom good	3.5	5.5	4.5	17.5	5.5	4	325	41.5	29	51
19	Bottom uneven	5.5	4	3	20	13.5	8.5	650	54.5	30	53
12	.....	3.5	5.5	3	21	7.5	4	215	44.5	31	55
11	Roll 18 ins. high	6.5	3.5	5	17	10	9	460	51.5	30	53
9	Cut through same	4	4.5	4	18	4.5	6.5	560	37.5	28	49
Entry	Bottom even	3	5	5	7	5.5	4	125	39.5	9	12
20	.....	39	36.5	35.5	130.5	58	50	2,505	5 hrs. 43 m.	216	377

THE SARGENT DRAFT OR PRESSURE GAGE.

In testing boiler and ventilating plants it is often important to have an accurate indication of the chimney draft, air pressure, etc., and the gage shown in the accompanying cut has been invented for this particular service. It may be used not only by the operating engineer to ascertain the chimney draft, but also by a ventilating engineer to indicate the pressure or vacuum maintained where large volumes of air are moved. It may also be used in testing gas engines for their

thermal efficiency, to show the pressures of the gas. For such purposes as the above an ordinary U-shaped glass tube is often used, the difference of the level of the liquid in the two legs indicating the pressure. It is claimed, however, that this can seldom be read closer than ¼-in. on account of the movement of the liquid. For closer readings, a glass tube has been used in an



An Easily Read Draft Gage.
C. E. Sargent, Chicago, Inventor. Schaeffer & Budenberg, New York, N. Y., Makers.

inclined position, giving 10 ins. travel for 1 in. rise, but the range for practical purposes is limited, as a tube showing a difference of pressure of 6 ins. would be 5 ft. long.

The necessity of accurate determinations and the desirability of a strong, portable, direct-reading, draft gage or manometer, which can be read to 0.01-in. and has a capacity of 6 ins. pressure or vacuum, led to the invention of the instrument shown herewith. It consists of a nickel-plated brass cylinder, closed at both ends, encircled with a helical groove in which is wound a transparent flexible celluloid tube, the bottom end of which is cemented in and communicates with the interior of the brass chamber. An extension of its lower head passes through the bracket which supports the gage; the cylinder may be revolved at will around its vertical axis, so that the level of the liquid comes on the front side. A small hose cock to which a rubber tube can be attached, admits pressure through the top head of cylinder. Distilled water, usually colored, is put in the cylinder through this cock until the zero mark on the scale is reached. The tube is tough and elastic and is not liable to be broken.

Pressure will cause the liquid to ascend in the tube, and for every inch of vertical rise it will travel around the cylinder, a distance of about 9 ins., which is divided into 100 equal parts. The diameter of the tube is such that the plane of the surface of the liquid is normal to the tube, making close reading possible. Rarefaction causes the liquid to descend, and by adjusting the cock, any degree of steadiness of the level in the tube may be obtained. The upper end of the tube is closed with a screwed brass plug and the inlet cock is shut when the gage is carried about, maintaining the liquid in place without danger of spilling.

The instrument is about 3 ins. diameter and 12 ins. long, weighing about 3 lbs., and it may be mounted on a special board or attached to a gage board. It was invented by Mr. C. E. Sargent, M. Am. Soc. M. E., First National Bank Building, Chicago, and is manufactured by Schaeffer & Budenberg, of New York.

COAGULATION, SEDIMENTATION AND UPWARD FILTRATION OF WATER AT BARTLESVILLE, I. T.

A small water purification plant is being built by the Bartlesville Water Co., of Bartlesville, I. T., as part of the water-works which is now under construction. Mr. W. P. Bullock is engineer of the company and has supplied us with the local information on which this article is based.

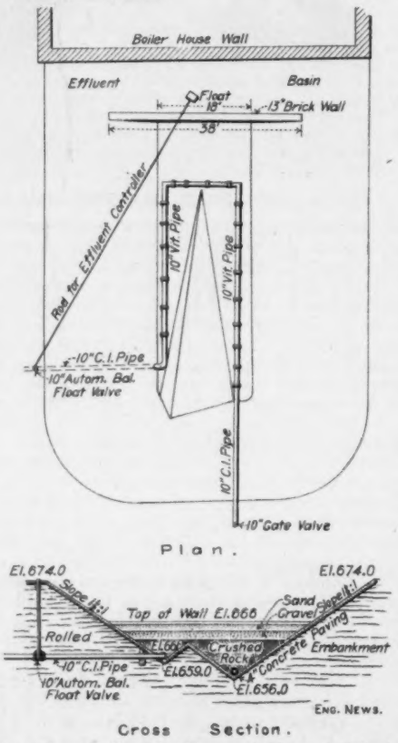
The water supply will be taken from Caney Creek, which is very muddy during the season of cultivation, the land comprising the drainage area being of a yellow, loamy soil and charging the water at such times with 1,100 parts per 100,000 of matter.

The population of Bartlesville is about 3,500. The pumping equipment will include two 8 x 10 x 12-in. simple duplex low-service pumps, to deliver the creek water to the coagulating and settling reservoir, and two 12 x 10 x 12-in. compound duplex pumps for the treated water. In addition, there will be a fire service pump.

At the suction of the low-service pumps sulphate of alumina will be admitted to the water being drawn from the creek and pumped to the coagulating and settling basin. The basin in question is about 60 x 80 ft. in plan at the bottom, has inner slopes of 2 to 1, and a depth of only 10 ft. to the top of the embankment.

A flexible outlet pipe is arranged to draw the water from 18 ins. below the water surface of the basin. This pipe leads to the bottom of the adjacent upward filter, the arrangement of which is shown by the accompanying plan and cross-section. The filter has an area of 0.05 acre.

It is calculated that 90% of the suspended matter will be deposited in the settling reservoir, during the 84 hours rest allowed, and that the remainder will be practically all removed by the upward filter. The peculiar shape of the bottom of the filter is designed to aid in the distribution of the water throughout the lower part of the bed, and also to facilitate washing the filter bed by reversing the flow.



Water Coagulating, Settling and Upward Filtering Plant at Bartlesville, I. T.

Mr. Bullock expects that the bulk of the dirt retained by the filter will remain below the sand since the effluent water is lighter than the unfiltered water. He also states that there is less likelihood of fouling the whole sand bed by admitting the water from the bottom against gravitation instead of with it.

Upward filtration, both in England and America, was tried three-quarters of a century ago, and has been tried at intervals since, but in general it may be said that it has not been found advantageous. Mr. Bullock suggests that perhaps in connection with coagulation, which will form a scum on the lower stratum of sand, upward filtration may prove successful. Sedimentation, assisted by coagulation, has also been employed for many years, and with success.

SHAFT SINKING THROUGH QUICKSAND AT THE SUSQUEHANNA MINE; HIBBING, MINN.*

By H. B. Sturtevant.†

The satisfactory completion of any engineering or mining feature is of interest to this institute; and further the institute has certain rights to information of the details of such accomplishment. Hence this paper on sinking, partly in quicksand, of the Susquehanna shafts at Hibbing, Minn., during 1903. Our conditions were in brief as follows: a surface of 138 ft., of which 57 ft. were in dry sand, 50 ft. in quicksand, and 31 ft. in hardpan.

Anticipating quicksand, it was decided to sink first, a small shaft, to reduce difficulties to a minimum; this shaft would drain the surrounding territory, making it less difficult to sink a large working shaft near by. And further the small shaft would be used as the permanent drainage and timber shaft. On Feb. 15, 1903, the sod, or rather frost, was broken for the small shaft of size 8 x 10 ft. inside; timbers 12 x 12 ins., studdles 3 ft. and hanging bolts 1¼ ins. The shaft bearers were 20 to 24 ins. diameter and 32 ft. long. At the depth of 45 ft. six ¾-in. wire ropes were used to hang the shaft from the bearers, using 8 x 8-in. wrought washers ½-in. thick.

At a depth of 57 ft., water and quicksand were encountered; and as considerable pull on the wire ropes was noticed, we built two trusses at the collar of the shaft using the bearers as lower cord of an ordinary queen post truss. From 57 ft. we used the drop shaft method. A shaft set was beveled to an edge flush on outside of shaft and placed in position and forced down; filling in other sets above it and under the jacks successively, bolting timbers to each other, and hoisting the sand, and pumping the water. In this manner we sank 13 ft., but could go no further and the operation was repeated until a depth of 85 ft. was reached. The experience was valuable, and we modified the method later in sinking the operating shaft, to be mentioned further on.

At 85 ft. depth, the pressure of sand being very great, and difficulties increasing, we reduced the size to 6 x 8 ft., building up inside the shaft a solid box with bottom edge beveled, but flush on outside, and this solid box was forced down 22 ft. to 107 ft. and bedded in hardpan. At 85 ft. the timbers were anchored to the collar trussed bearers by four 1¼-in. ropes, thus making ten wire rope hangers, besides the usual iron hangers. At 107 ft. the timbers were again anchored by wire ropes. The enormous strain on the shaft timbers due to quicksand was manifested by the washers 8 x 8 ins. pulling into the bearers fully 2 ins.

During the entire process we forced in advance of the timber a sheet steel box, 3 x 5 ft. of ¼-in. plates, put together with angle irons, and 10 ft. long. This box was forced down in the middle of the shaft and served to partially drain the bottom, and to serve as a sump for the pump suctions.

Progress in quicksand was very slow, and 8 to 10 ft. per month was all we could do, the rate varying from nothing to 6 ins. per day. We found as others have, that the great danger to success lies in the sand and water bubbling through under the bottom timbers. To prevent this, the sand was not removed faster than necessary, and when bubbling was indicated a plank hewn to edge on end, or a shovel was used and water allowed to rise a few feet; further, during the entire work in quicksand we ran eight-hour shifts, work progressing continuously in order to hold what we had made. And to avoid trouble over the idle Sunday, the sand was not removed at bottom and water was allowed to rise to the pumps.

In hardpan below 107 ft. depth the process used was driving lath, removing ground, and placing timbers, bolting them together. Sinking progressed in clay at the rate of 1½ ft. per day, until the shaft was bottomed on ledge of ore at 138 ft. on Sept. 14, or seven months from date of beginning.

The drainage increased to 300 gallons per minute on July 1, at 90 ft. depth and 450 gallons at 138 ft. Duplex sinking pumps were tried but were unsatisfactory, because of jumping in case of air in the suction; and No. 10 Cameron pumps were substituted.

The shaft is only 2 ft. out of plumb; and this does not impair its serviceability for pumping and timber. In sinking it was found necessary to use the greatest care to maintain the bottom edge of timbers exactly level in quicksand or the shaft would sink out of plumb. The water would occasionally make cavities which would fill sud-

*Abstract of a paper read at the meeting of the Lake Superior Mining Institute, Ironwood, Mich., Aug. 16, 1904.

†Manager of Mines, Buffalo & Susquehanna Iron Co., Hibbing, Minn.

denly, producing a great strain on timber and hanging ropes. There was no way of anticipating this, and as the timbering was solid there was no way to inspect behind it. Anticipating these troubles we would, while sinking, ram hay behind the timbers, and the effect was to prevent dirt washing down on outside of shaft.

It is also very necessary in sinking in quicksand to hoist no more sand than possible; that is to say, the sand will try to run into the shaft under the timbers, and if removed too fast the sand outside the shaft will become loosened and run, and cause timbers and irons to break; and the whole work may be ruined irreparably. On one occasion we noticed unusual bubbling and we immediately stopped the pumps and allowed the water to rise 15 ft.,

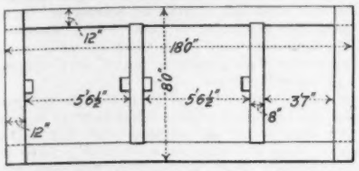


Fig. 1. Plan of Hoisting Shaft Sunk through Quicksand: Susquehanna Mine; Hibbing, Minn.

and then lowered sand from the surface and filled the shaft several feet in which condition it stood, efforts to resume being without success for six days, when sinking was resumed.

The hoisting shaft was begun July 28, 1903, 4 ft. east from the drainage shaft. The size was 6 × 16 ft., inside, with usual timbering, to the water line. It is shown in plan in Fig. 1. It was found that the drainage shaft pumps had drained the ground at the hoisting shaft to such an extent that a No. 7 Cameron working slowly handled the water.

The same difficulties were encountered as at the drainage shaft, but in less degree, and we are satisfied that had we not partially drained the ground by the small shaft, it would have been impossible to sink the large one, because the troubles are in proportion to the external dimensions. The method in the quicksand in the hoisting shaft was quite different from that used in the drainage shaft; and particular attention is called to the following details, as we consider the method a most successful one, and not, so far as we can learn, one which has before been used.

A drop section of shaft timbers of full size of shaft was framed and placed in position. The details of the drop section are shown in Fig. 2. It is 4 ft. in depth, made of 12 × 12-in. timbers bolted together, and braced; the lower two sets are beveled, flush outside, and a ⅛-in. plate of steel-shaped to fit the edge was fastened by spikes, making a shoe or edge 3 ins. wide to cut the sand; outside of this drop section was bolted a sheeting of plank, with lower edges hewn thin for less resistance to sand, and the upper ends of plank extending 3 ft. 6 ins. above the timbers, as indicated in sketch, and behind the last permanent shaft timbers.

About 20 or 30 jacks, 2½ × 18 ins., were operated between the drop section and the last permanent timbers, the plank sheeting serving to retain the sand. As the jacks forced down the drop section they were removed tempor-

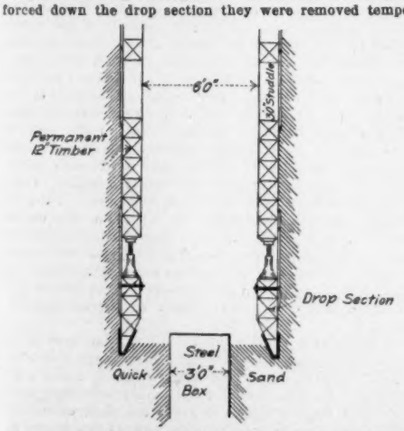


Fig. 2. Section of Shaft Sunk through Quicksand.

arily, and timbers placed, and bolted to the last permanent timbers; and to bring the permanent timbers close together pockets were cut out for nuts and washers. In other words, as the jacks forced down the drop section the permanent timbers were placed above the jacks and this process was continued until the iron shoe was imbedded in clay at 107 ft., after which we drove lath and after removing the ground put in timbers.

As this was to be the hoisting shaft it was very necessary that it be vertical and truly in line without serious variation. The greatest possible care in the use of jacks was necessary to success in this particular. The greatest variation from vertical is only 6 ins. As an additional protection against a crooked shaft we placed diagonal braces 8 × 8 ins. at each end from the collar of the shaft down to depth of 85 ft. The necessity for these braces might be questioned, but our experience in the drainage shaft, where the washing away of sand behind the timbers and the caving causing the entire shaft to surge several inches, prompted us to take no chances. The shaft timbers were hung by wire ropes from the bearers at collar of shaft, but it was not necessary to truss the bearers as in the case of the drainage shaft.

As an improvement I might suggest the substitution of steel sheeting for the plank as less liable to break or get loose;—and further the steel would, being thin, allow the drop section to cut the sand easier.

We sank the shaft through the quicksand at the rate of 8 ft. 6 ins. to 9 ft. 6 ins. per month, working continuously, excepting Sundays. On reaching hardpan on Jan. 1, 1904, we sank at the rate of about 1½ ft. per day. As to speed of sinking in quicksand we shall not boast, but we modestly claim success in continual progress and that we did not lose either shaft; neither did we lose headway gained, excepting on the occasion mentioned above, and further our hoisting shaft has a variation of only 6 ins.

For our conditions and with 50 ft. of quicksand I can suggest no improvement over the methods used at our hoisting shaft. However, if the quicksand were 100 ft. deep, the difficulties would increase and it is probable that the freezing process might be more successful, and more rapid; although the method described could be used. The essential feature of our success was the solid drop section with steel shoe and with back lath well secured and extending upwards to span the open space occupied by the jacks.

To Capt. Thos. Caddy I am pleased to give great credit for success in the sinking of the shafts.

RAISING A 7,500-TON SCHOOL BUILDING.

RAISING A FOUR-STORY BRICK BUILDING.

The raising of a brick, four-story school building, weighing 7,500 tons is illustrated by the accompanying photograph. The building is 84 × 150 ft. in size, located at Covert St. and Evergreen Ave., Brooklyn, and was raised 34 ins. in order to have the basement above flood water level. Work was begun, preparatory to the raising, on June 4, and the actual raising was begun July 15 and completed July 23. A trench, 8 ft. wide, was dug around the building to the depth of the foundation, and in this trench were placed yellow pine timbers running parallel with the walls. Above the entrenched timbers, and on each side of the foundation, cribbing blocks were built up upon which jack-screws were placed, and resting thereon were the "screwing timbers" upon which were placed the cross timbers and top-sticks, the last named carrying the beams of the building. Seventy-five men were sent into the foundation and were there distributed into small gangs upon the different sections of cribbing, each gang patroling its own section. In single file the men followed each other, giving each of the 1,200 jack-screws a quarter turn. In this manner the massive structure was lifted safely to the desired height and securely held until the foundation was built under it. This is said to be the heaviest building ever raised in Greater New York. In any case it is a feat of engineering well worthy of being placed on record. The contract price of the work is said to have been $25,000. Miller, Daybill & Co., of 318 Kosciusko St., Brooklyn, were the contractors.

TESTS ON REINFORCED-CONCRETE BEAMS.*

By F. E. Turneaure,† Assoc. M. Am. Soc. C. E.

SCOPE AND OBJECT OF THE TESTS.—During the year 1902-3 a series of experiments on reinforced-concrete beams was made in the laboratory of the University of Wisconsin, from which the most important result obtained related to the early appearance of cracks in the concrete. To verify these results and to carry on studies in other directions a second series of tests was made during the past winter. These tests were conducted under the direction of the author by Messrs. J. M. Gilman and G. E. Kahn as a thesis.

It has not been possible to work up the data of these tests completely for this paper, but it has been thought worth while to present the results as far as they concern (1) the early cracking of the concrete on the tension side; (2) the position of the neutral axis at different stages of the test; and (3) general results as to strength of beams reinforced by different methods and loaded in different ways.

DESCRIPTION OF TEST BEAMS.—Table No. 1 gives a comprehensive statement of the character of the principal test beams. The concrete, as noted, was a 1: 2: 4 mixture, all ingredients being proportioned by weight. Vulcanite Cement was used, good coarse sand, and a crushed limestone of ¾-in. size and smaller, the very fine dust being screened out. Enough water was used to make a mass easily put in place with a very small amount of tamping. All these beams were 6 × 6 ins. in section and about 64 ins. long (span length 60 ins.). The reinforcing steel rods were spaced ¾-in. to 1-in. from the bottom. Where bent rods were used the bend occurred 10 ins. from the center, beyond which point the rod was carried up to within ¾-in. from the top of the beam at the support. Stirrups were made of 3-16-in. round rods and spaced in pairs 3 ins. apart except at the center, where for a distance of 12 ins. no stirrups were used.

All beams were left in the molds for 48 hours, during which time they were covered with wet cloths. After removal from the molds they were stored in running water at a temperature of about 60° F. until about four hours before testing. The particular object of thus storing the beams was to enable minute cracks to be more readily detected, in accordance with our experience of the previous year.

*A paper presented before the American Society for Testing Materials, at Atlantic City, N. J., June, 1904.

†Dean of the College of Mechanics and Engineering of the University of Wisconsin, Madison, Wis.

COMPRESSIVE STRENGTH OF CONCRETE.—Compressive tests were made of cubes of each batch of concrete used. Three-inch cubes were tested generally, and additional four-inch cubes in some cases. The results as to strength are given in Table III. The measurements of distortion proved very unsatisfactory, so no values of the modulus of elasticity can be given.

TESTS OF THE STEEL USED.—Table II gives results of tensile tests on the different kinds of steel used. The elastic limit was determined by the "drop-of-beam," checked by the use of dividers on the test specimen. A plain bar was purposely selected of a very high elastic limit.

APPARATUS AND METHODS OF TESTING.—About one-half of the beams were tested with a center load and one-half with two equal concentrated loads placed at the third points, thus giving a uniform bending moment over a length of 20 ins. The beams tested by the first method were one-month beams, those tested by the other method were three-month beams. All beams were tested with the tension side uppermost in order that observations for cracks could be readily made.

In both series the central deflection was carefully observed by micrometer measurements on both sides of the beam. In the second series measurements were also made of longitudinal deformations at top and bottom over a length of 14 ins. The apparatus for the latter purpose was specially constructed. It consists of two sets of frames clamped against the sides of the beam, one set carrying four graduated dials with pivoted pointers. Very fine copper wires are made fast to posts on the other frames and led over small pulleys attached to the pointers. Small weights serve to keep the wires taut and to furnish the necessary friction to operate the pointers. The dials are graduated to read .001-in. directly and .0002-in. by estimation. Distortions are thus measured on four lines. This apparatus proved very satisfactory and was entirely reliable.

Careful observations for cracks were made and our experience of the previous year enabled them to be detected at a very early stage. It had been found that, by testing the beams when somewhat moist, a crack is made visible when exceedingly small, it appearing first as a narrow, wet streak perhaps ⅛-in. wide and a little later as a dark hair-like crack. It was not necessary to search for the lines with a microscope as under these conditions they were readily found.

That the wet streak, called a "water-mark" hereafter, shows the presence of an actual crack was demonstrated last year by sawing out a strip of the concrete containing such a water-mark; the strip fell apart at the water-mark.

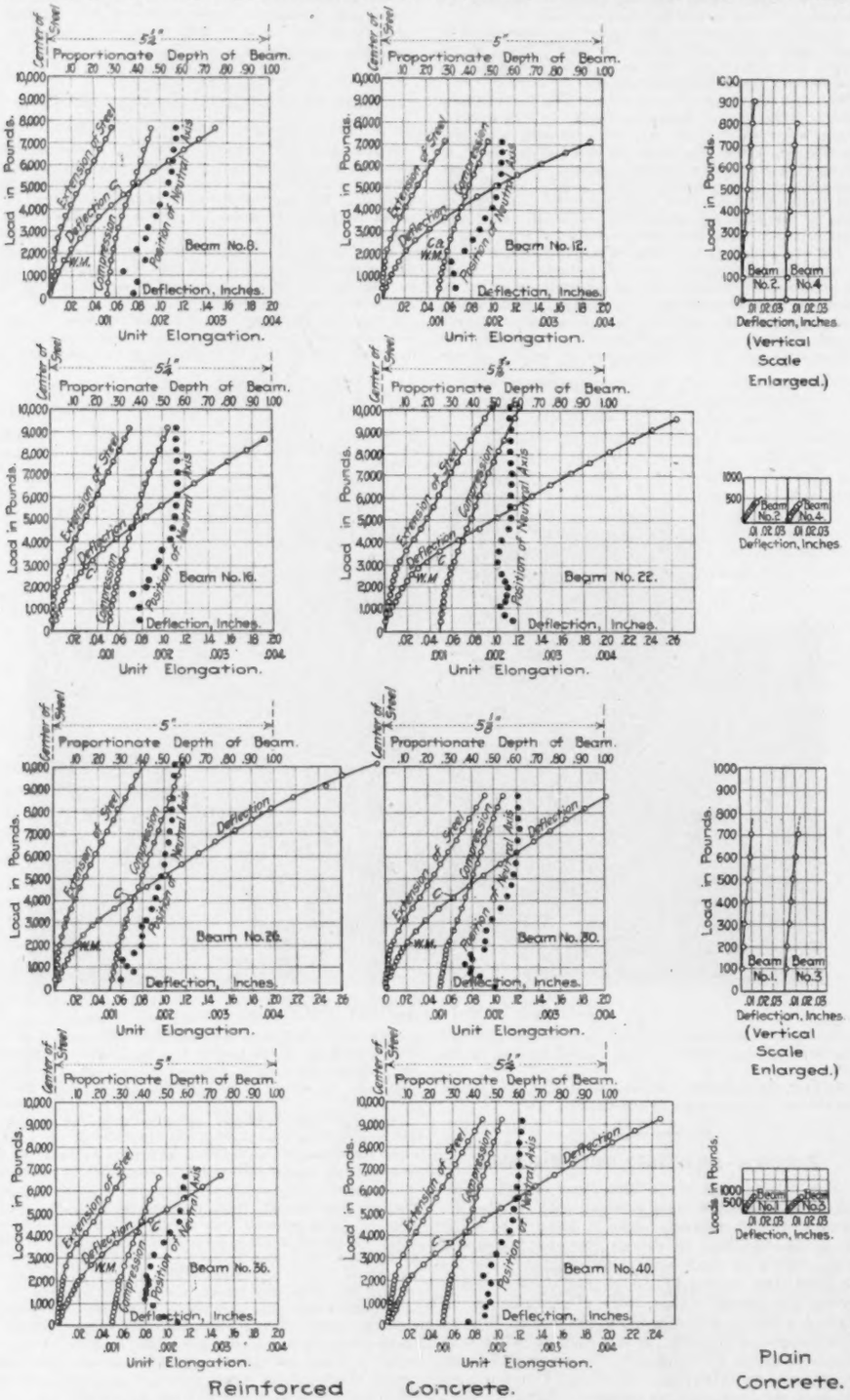
RESULTS OF THE TESTS.—Tables IIIa and IIIb, and the several diagrams, give the principal results of the tests. In Table IIIa are given the loads at rupture, for three-month beams tested with two loads, and the observed extension of the extreme fiber of the concrete at the appearance of the first water-mark and also at the time when the first crack became plainly visible as a crack. The compressive strength of the concrete is given as determined from the average of the tests. Table IIIb, for one-month beams under single load, gives the same information excepting that the extensions are here calculated from the deflections, assuming the form of curve of the beam to be the same as in the usual theory of flexure and the neutral axis to be in the center of the beam at this stage of the test. In the diagrams are plotted the deflections, the proportionate extension of the steel and compression of the extreme compressive fiber, and the deduced location of the neutral axis assuming plane sections, for one beam of each kind tested. The appearance of the first water-mark and of the first crack are also noted on the deflection curves.

It will be noted from these tables that the unit-elongation of the concrete at the time of the first visible crack is in many cases as small as .00035, which is considerably less than that given by previous observers. And, if we take the water mark as indicating the presence of a crack not yet visible, the elongation of the concrete is seen to be usually between .00010 and .00020.

The calculated elongations of Table IIIb are subject to some uncertainty, but for the early stages of the test they cannot be far wrong. A similar method of calculation applied to the three-month beams gives results agreeing very closely with the observed values up to a unit-elongation of about .0003. Above this the calculated values become too small owing to the change in position of the neutral axis. In general the first water-mark of Table IIIb was more carefully observed than in the tests of Table IIIa, as the extension apparatus interfered somewhat in the latter case. In the plain concrete no water-marks or cracks were observed before rupture. Comparing the observed and calculated elongations of the reinforced concrete with those for the plain concrete at rupture, it will be seen that the initial cracking in the former occurs at an elongation practically the same as in the latter. It will also be noted on the diagrams that the initial cracking as shown by the water-marks usually begins about where the curves begin to change direction rapidly.

The significance of these minute cracks is an open question. It has been supposed that concrete reinforced by steel will elongate about ten times as much before rupture as will plain concrete. These experiments show very clearly that rupture begins at about the same elongation in both cases. In the plain concrete total failure ensues at once; in the reinforced concrete rupture occurs gradually, and many small cracks may develop so that the total elongation at final rupture will be greater than in the plain concrete. In other words, the steel develops the full extensibility of a non-homogeneous material that otherwise would have an extension corresponding to the weakest section.

The presence of these cracks of course seriously affects the tensile strength of the concrete, and, as they appear at an elongation corresponding to a stress in the steel of 5,000 lbs. per square inch or less, it would seem that no allowance should be made for the tensile resistance of the concrete. Furthermore, if such cracks are present the calculation of the tensile resistance of reinforced con-



Note.—Deflections and elongations are referred to the scales at bottom of diagrams. Position of neutral axis is referred to the scale at top of diagram.

All beams, 6 × 6 ins., by 90 ins. span, tested at three months (except Nos. 1 and 3, which were tested at one month). Beams 8, 12, 16, 22 and 40 were reinforced with plain round rods.

Beam 26 reinforced with Johnson bars.
Beam 30 reinforced with Ransome bars.
Beam 36 reinforced with Thacher bars.
Beams 1, 2, 3 and 4 had no reinforcement.
W. M. indicates the point where the first "water-mark" was observed.
C. indicates the point at which the first crack was observed.

DIAGRAMS SHOWING RESULTS OF TESTS OF CONCRETE AND REINFORCED-CONCRETE BEAMS AT THE UNIVERSITY OF WISCONSIN.

Curves of Deflections, Elongations, and Position of Neutral Axis.

TABLE I.—List of Beams Tested.
All Beams of 1; 2; 4 Concrete.
6x6 inches, by 60-inch span.

Serial Number.	No. of beams tested.	Kind of rods.	Size of rods.	No. of rods.	Method of reinforcing.	Reinforcement, per cent.
1 to 4	4	No rods	.....	....	.....	0
5 — 8	4	Plain round	½-in.	4	Straight	1.07
9 — 12	4	" "	⅜-in.	2	"	0.97
13 — 16	4	" "	½-in.	4	2 straight and 2 bent up.	1.07
17 — 20	4	" "	½-in.	4	All straight with stirrups	1.07
21 — 24	4	" "	½-in.	4	2 straight and 2 bent, with stirrups	1.07
25 — 28	4	Johnson	½-in.	2	Straight	.89
29 — 32	4	Ransome	⅞-in.	2	Straight	0.98
33 — 36	4	Thacher	⅝-in.	2	Straight	0.97
37 — 40	4	Plain round	½-in.	4	All straight with stirrups	1.07

TABLE II — Tension Tests of Steel

Kind of Rod	Nominal Size	Elastic Limit lbs. per sq. in. actual section	Ultimate Strength lbs. per sq. in. actual section	Elongation per cent.
Plain round	½ inch	75,000	105,000	19.4
Plain round	⅝ "	75,000	93,000	17.0
Thacher	½ "	45,000	62,000	16.2
Johnson	¾ "	59,000	92,500	18.7
Ransome	⅞ "	73,000	88,500	10.0

TABLE No. III-a.—Tests of Reinforced-Concrete Beams, under Two Equal Loads at Third-points.

Serial Number	Kind of Reinforcing Rods	Load at Rupture lbs.	Proportionate Elongation as measured		Compressive Strength of Concrete from Cube Tests lbs. per sq. in.
			at first water-mark	at first crack	
8	Plain	7,700	.00011	.00064	4,250
12	Plain	7,700	.00034	.00034	2,500
10	Plain	7,900	.00024	.00046	
14	Plain	10,200		.00056	2,800
16	Plain	10,700		.00032	
22	Plain	10,200	.00025	.00065	2,775
24	Plain	9,200	.00018	.00048	
26	Johnson	10,200	.00016	.00056	3,000
28	Johnson	8,700	.00013	.00040	
30	Ransome	9,700	.00012	.00064	2,600
32	Ransome	9,360		.00050	
36	Thacher	7,400	.00028	.00090	3,850
34	Thacher	7,340	.00025	.00050	
38	Plain	13,100		.00066	3,850
40	Plain	9,580		.00050	

All beams were three months old.

TABLE III-b.— Tests of Concrete and Reinforced-Concrete Beams, under Single Center Load.

Serial Number	Kind of Reinforcing Rods	Load at Rupture	Proportionate Elongation as Calculated		Compressive Strength of Concrete
			at first water-mark	at first crack	
2*	None	1,000		.00018	3,000
4*	None	900		.00011	
3	None	800		.00014	2,500
1	None	800		.00010	
7	Plain	8,100	.00015	.00056	3,500
5	Plain	5,700	.00020	.00031	
11	Plain	5,000	.00014	.00018	2,200
9	Plain	5,500	.00011	.00048	
15	Plain	7,300	.00007	.00011	2,350
13	Plain	6,400	.00009	.00011	
23	Plain	5,500	.00020	.00060	2,500
21	Plain	6,250	.00027	.00049	
27	Johnson	6,400	.00011	.00031	2,300
25	Johnson	6,460	.00009	.00048	
29	Ransome	8,125	.00019	.00100	2,000
31	Ransome	7,880	.00012	.00083	
35	Thacher	5,900	.00013	.00053	3,150
33	Thacher	5,200	.00021	.00095	
39	Plain	8,250	.00023	.00053	2,740
37	Plain	6,700	.00010	.00033	

*Three months old.
All other beams were one month old.

TABLE IV.—Moments of Resistance of Beams, and Stress in Steel.

Serial Number.		Actual Moment of Resistance. Inch-pounds.	Area of Steel. sq. inches.	Calculated Stress in Steel at Rupture of Beam. Pounds per sq. inch.
5,7.	Plain	104,000	0.385	61,000
8	Plain	77,000		46,000
9,11	Plain	79,000	0.35	52,000
10,12	Plain	78,000		52,000
13,15	Plain	103,000	0.385	61,000
14,16	Plain	104,000		61,000
21,23	Plain	88,000	0.385	52,000
22,24	Plain	97,000		57,000
25,27	Johnson	96,000	0.32	70,000
26,28	Johnson	95,000		70,000
29,31	Ransome	120,000	0.35	78,000
30,32	Ransome	95,000		62,000
33,35	Thacher	77,000	0.35	52,000
34,36	Thacher	74,000		50,000
37,39	Plain	112,000	0.385	65,000
38,40	Plain	113,000		65,000

crete by the method used by Considère leads to no useful result. In his tests Considère determines the stress in the steel from measurements of its elongation and then assumes the concrete to carry the remainder of the load. Assuming the value of E to be uninfluenced by the concrete, this would be correct so long as the stress in the steel and in the concrete is uniform between points of measurement. As stated by Considère himself, such results are only average values. But the concrete may be cracked entirely through and yet possess a very considerable average tensile strength over a length of several inches. Obviously in that case an average is of no value; the strength of the concrete is really zero.

In practical design the most important question which arises is how far a concrete may be cracked without exposing the steel to corrosive influences. In this respect it seems to the writer that the minute cracks which appear in the early stages of the tests can have very little influence. However, the entire question of the effect of cracks and pores in the concrete on the corrosion of the steel needs careful investigation.

POSITION OF THE NEUTRAL AXIS.—The diagrams show the neutral axis to lie at first very near the center of the concrete beam. As the cracks develop it moves gradually nearer to the compression side. It should be noted that the neutral axis as here found, by measuring deformations over a length of 14 ins., is the average neutral axis over this distance. At the point where a crack exists, the elongation per inch of the steel is more than elsewhere and the neutral axis is therefore nearer the compression side than is the average position.

ULTIMATE STRENGTH OF THE BEAMS.—In Table IV are given the ultimate moments of resistance of the beams and the resulting stresses in the steel as determined by calculation. In this calculation the center of pressure of the concrete is assumed to be five-eighths of the distance from the observed neutral axis to the outside surface, corresponding to a parabolic variation of compressive stress. (The assumption of a straight line variation would change the calculated values only about 2%.) The average of the two results for the one-month beams is given first in each group and that for the three-month beams just below. These calculations indicate that the full elastic limit strength of the Johnson and the Thacher bars was developed and probably of one or more of the Ransome bars, but that in the other cases failure occurred before the elastic limit was reached.

The stresses in the bars as here calculated, which cannot be greatly in error, will be seen to be in nearly every case much higher than would be deduced from the observed elongations of the steel as given in the diagrams, using the usual modulus of elasticity. Thus in the case of beam No. 12 the elongation at rupture was .00115. Assuming a value of E of 29,000,000 the stress would be 33,000 lbs. per square inch, whereas the calculations from the actual moment give a stress of 53,000 lbs. The discrepancy is doubtless due partly to the fact that the calculations from the elongations cannot give us the maximum stress on the rods, but it is impossible that all of the difference can be accounted for in this way. There must have been a large initial tension in the rods which of course would not be indicated by the extensometer readings. As the beams were hardened in water such initial tension is very probable.

In but a few cases was the failure free from the influence of shearing stresses, the rupture usually occurring outside the load and on a diagonal line. In a few cases, after the cracks had opened up well, the concrete failed in compression. The maximum compressive stress in the concrete, calculated on the assumption of a parabolic law, ranged from 2,100 lbs. per sq. in. for the weakest beam to 3,000 lbs. per sq. in. for the strongest. No trouble was experienced from slipping of the rods, except in one case where they had become displaced in molding.

THE DECISION IN THE SEPTIC TANK INFRINGEMENT SUIT AGAINST PAWTUCKET, R. I.

The essential features of the sewage patent decision in favor of the city of Pawtucket, R. I., and against the American Sewage Disposal Co., of Boston, noted briefly on p. 202 of our issue of Sept. 1, 1904, were stated in Judge Brown's decision in the U. S. Circuit Court as follows:

The principal question in the case relates to that element of the claim described as "a series of stationary primary filtering beds constructed to discharge the effluent wholly through filtering material."

The defendant uses a primary tank which contains no filtering material, but which discharges the effluent upon outside oxidizing filter beds by means of a pipe. The defendant's tank at times has been used for putrefactive action, to liquefy and dispose of the solid matter of sewage. It is charged that the defendant thus uses the substance of the complainant's patent, namely, putrefactive action followed by oxidation for the complete disposal of sewage.

In my opinion the defendant does not infringe for the reason that it does not have the element of the claim, "a series of stationary filter beds constructed to discharge the effluent wholly through filtering material," nor any equivalent therefor which will perform the function of filtering the sewage to separate the solids and liquids. The use of putrefactive action to liquefy solids is not an infringement of anything described or claimed in the patent, or proved to be substantially involved in any operation of the patented structure intended by the patentee.

It does not seem necessary or useful to consider the defense of anticipation of the claim as construed by the complainant, since I am of the opinion that the complainant's construction of the claim is unsound.

The works for the treatment of the sewage of Pawtucket were described in our issue of Jan. 2, 1896. The plant includes settling tanks and intermittent filters. For ten months in 1900 one of the tanks was used as a septic tank (see Eng. News, Dec. 20, 1900), and in 1900-1 the tank was again used for that purpose, some of the effluent being applied to experimental so-called bacterial beds (see Eng. News, Feb. 27, 1902). It appears that the complainant in this suit based the infringement claims on the use of the settling tank as a septic tank, whereas the patent, as stated in the decision, is for a filter bed in combination with a second bed. The substance of the patent specification and claim was given on p. 133 of our issue of Aug. 11, 1904.

THE PROPOSED GUNNISON TUNNEL AND SOUTH CANAL, COLORADO.

A tunnel 30,000 ft. long to deliver 1,300 cu. ft. of water per second from the Gunnison River, Colo., to the Uncompahgre Valley, is now advertised for letting by the U. S. Reclamation Service. The character and magnitude of this work are such as to be of more than passing interest.

Fig. 1 shows the location of the proposed tunnel and the proposed canal into which it will discharge the water diverted from the Grand Canyon of the Gunnison. Fig. 2 gives the profile of the mountain to be penetrated, from which it appears at a glance that the tunneling must be carried on from only two headings. Since bids are to be opened Oct. 5, 1904, and since the tunnel must be completed April 15, 1908, it is evident that even if the contract is awarded by Oct. 15 there are only 42 months in which to drive and line nearly 30,000 ft. of tunnel; from this, of course, must be deducted the time required to install a plant, so that a rate of speed well above the average records for heading work in this country must be maintained.

Cross-sections of the tunnel are shown in Figs. 3 and 4. About 4,000 ft. of the tunnel will be timbered and lined, as in Fig. 4; about 5,000 ft. as in Fig. 3; about 11,000 ft. as in Fig. 3, except that the minimum thickness of concrete will be 6 ins.; and about 10,000 ft. as in Fig. 3, except that the concrete arch will be omitted.

Reverting again to the time of completion, it may be noted that a forfeit of $2,000 for each day of non-completion, for a period not exceeding 30 days, will be exacted, and a similar bonus will be given for completion within the time limit. It is safe to predict that this bonus provision will

spur the contractor to win the prize offered, not only because of the money involved, but because of the honor felt in winning.

The contractor will be required to pump 10,000 gals. per 24-hour day without extra charge, but any excess of water will be paid for at the rate per 1,000 gals. specified in the proposal.

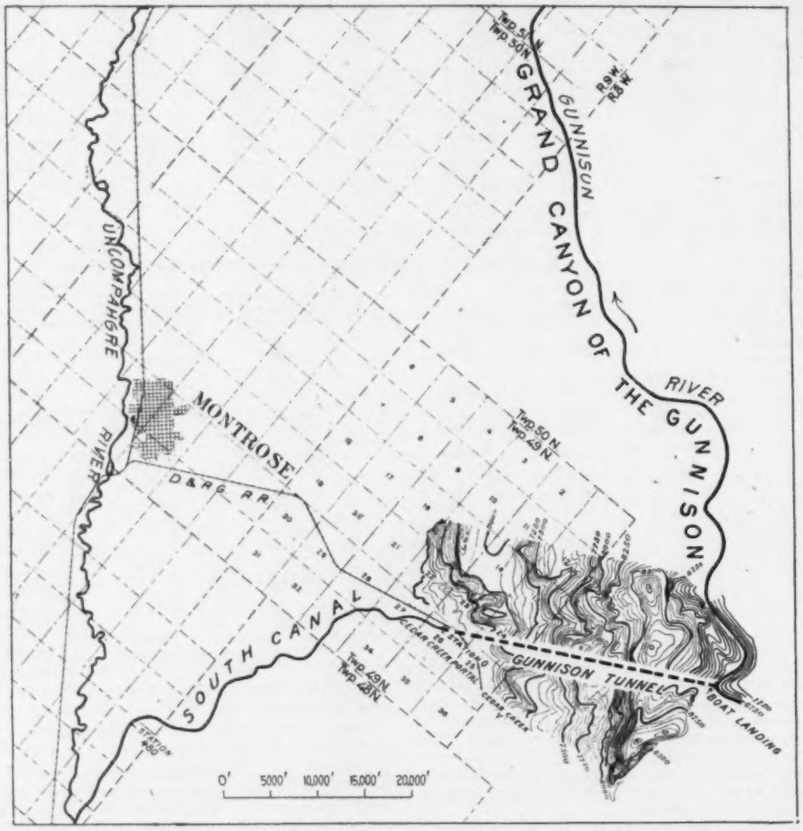


FIG. 1. MAP OF GUNNISON TUNNEL AND SOUTH CANAL IN UNCOMPAHGRE VALLEY, COLORADO.

The specifications state that

> Any excavation beyond the neat lines of the tunnel section, as shown on the drawings, shall be filled to the satisfaction of the engineer in charge, and no allowance will be made therefor.

But it is not clear whether this filling may be broken stone "packing," or whether concrete will be required. The contractor is, of course, left in doubt as to how much of this filling will be necessary until he learns by actual experience how the rock will break out.

The excavation of the South Canal, Fig. 1, is divided into nine "divisions," the right being reserved to award as many or as few divisions to one bidder as may seem best. There are about 443,000 cu. yds. of excavation in eight miles of the South Canal, of which 324,000 cu. yds. are plowable earth, 84,000 cu. yds. of indurated material, and 35,000 cu. yds. of loose rock. The free haul limit is 200 ft., and there is very little overhaul shown in the bidding sheet.

TESTS OF STEEL-CONCRETE BEAMS AND THEIR MATERIALS.*

By Edgar Marburg † M. Am. Soc. C. E.

A series of tests on steel-concrete beams has just been completed in the testing laboratory of the Civil Engineering Department of the University of Pennsylvania. These tests were made the subject of a joint thesis on the part of four members of the senior class, Messrs. George Freeman, Jr., Thomas Hovenden, H. M. Peirson, and A. C. Toner. The program was planned and executed under the direction of the writer.

SCOPE OF TESTS.—The beams were made and tested under conditions as nearly identical as practicable in every respect, except span and character of reinforcement. Twenty-five 8 × 8-in. beams of 1, 2, 4 broken-stone concrete were tested at the age of 30 days under central loading: twelve on an 8-ft. span, thirteen on a 5-ft. span. The lengths, out to out, were 8.5 and 5.5 ft. The concrete was both plain and reinforced, the reinforcement consisting of plain, square rods, and rods of the Johnson, Ransome and Thacher types. The rods were nominally ½-in. square, although the actual net sections varied from 0.18 to 0.255 sq. in. for the various types. Three rods were imbedded in each beam with centers 1 in. from the bottom surface. Laterally the two side rods were centered 2 2-3 ins. from the middle rod. The percentage of metal to the total cross-section varied from 0.84 to 1.19%. Various collateral tests were made to which further reference will be had hereafter. The specimens for these tests were all of the same composition as the beams, and were tested at the same age, 30 days.

MATERIALS.—The cement used was Atlas Portland, purchased on the open market. The tensile strength, neat, in pounds per square inch, was 337 at 1 day, 629 at 7 days, and 793 lbs. per sq. in. at 30 days, each value being the average of 6 tests. The mean departure from the average value was 3.8, 4.5 and 3.4%, respectively, for the three sets. The determination of fineness showed the following residues: on No 50 sieve 0.5 %, on No. 100 sieve 4.8%, on No 200 sieve 22.4%. The time of initial set, determined by means of the Vicat needle, ranged from 23 to 25 minutes for three observations. Pat tests in air and water gave normal indications.

The stone was ¾-in. trap-rock from Glen Mills, Pa., and gave the following residues: on sieve of ¾-in. mesh 0.64%, on sieve of ½-in. mesh 35.5%, on No. 6 sieve 92.5%.

The sand used was a good quality bar sand from the bed of the Delaware River at Bordentown, N. J., and gave the following residue: on No. 20 sieve 2.8%, on No. 30 sieve 11.8%, on No. 50 sieve 59.6%.

MIXING AND MOLDING.—The materials used for the concrete were proportioned by measure: 1 part of sand to 2 parts of cement and 4 parts of broken stone. To avoid variations in the quantities of cement due to differences in the degree of compacting, the standard volume was weighed and the quantities were afterwards determined by weight. The cement and sand were thoroughly mixed dry by shovel to an even color, and 16% (of their combined weight) of water was then added. The mortar was turned until it was of uniform consistency, and the broken stone, previously dampened, added. The concrete was turned until the stones were thoroughly coated with the mortar. The concrete was rammed into the molds by means of a 27.5-lb. rammer with a flat bottom face 6 × 8½ ins. Great care was taken to insure accuracy in placing the rods. The thickness of the bottom layer was fixed by means of a template. The rods were then introduced, concrete added, and gently tamped, more concrete added and thoroughly rammed as above described.

The water rose freely to the top during ramming. A wet mixture was chosen to insure solidity and uniformity of the concrete and the adhesion of the imbedded rods. No voids appeared in the fractured surfaces of any of the broken specimens, and the bond between the rods and the concrete was continuous and apparently perfect.

The specimens were stored in the laboratory at a temperature of about 70° F. The 8-ft. beams were left in the molds for about a week, the 5-ft. beams for about three days. No measures were taken to keep the surfaces damp during hardening, and no visible shrinkage cracks developed. The weight of the concrete was about 154 lbs. per cu. ft.

Wooden molds of two types were used: The one hinged longitudinally along a central line at the bottom, the other with one removable side-piece. The latter proved more convenient. In both types the end-pieces were detachable.

Specimens were also prepared for compression, tension

TABLE I.

No. of beam.	Method of Failures.	Description of Failure.
P^8_1	1	Diagonal crack at bottom, 12 ins. from center, running to center at top. Secondary crack running from first crack towards end in plane of rods.
P^8_2	1	Diagonal crack at bottom, 6 ins. from center, running to center at top. Rods slipped.
P^8_3	1	Diagonal crack at bottom, 15 ins. from center, running to center at top. Secondary crack running from first crack towards end in plane of rods.
P^5_1	1	Diagonal crack at bottom, 11 ins. from center, running to center at top where crushing occurred.
P^5_2	4	Hair-line crack at bottom, 4 ins. from center, widened and ran a short distance up side; final failure occurred by crushing of concrete at top.
P^5_3	4	Similar to P^5_2; except that hair-line crack started at center.
J^5_1	3	Hair-line cracks at bottom, 10 ins. each side of center, ran up side a short distance. Final failure by sudden diagonal crack from center at top to end support.
J^5_2	3	Sudden failure by diagonal crack from center at top to end support.
J^5_3	1	Sudden failure by two diagonal cracks on same side of center, 15 and 22 ins. from the center, running from plane of rods to center at top.
J^5_4	1	Sudden failure by diagonal crack 18 ins. from center running from plane of rods to center at top, and crack in plane of rods from first crack towards end.
J^8_1	4	Many hair-line cracks along bottom. Concrete crushed at top. Secondary diagonal crack.
J^8_2	1	Several hair-line cracks on sides. Diagonal failure crack 12 ins. from center, running from plane of rods to center at top. Irregular cracks in plane of rods.
R^5_1	1	Diagonal crack 13 ins. from center, running from plane of rods to center at top.
R^5_2	1	Many hair-line cracks on bottom. After ultimate load was reached, diagonal cracks appeared 14 ins. on each side of center.
R^5_3	1	Sudden failure by diagonal crack 17 ins. from center, running from plane of rods to center at top. Secondary crack running from first crack towards end in plane of rods.
R^8_1	1	Irregular diagonal crack 22 ins. from center at bottom to center at top. Two large cracks across bottom 15 and 17 ins. from center.
R^8_2	4	Many hair-line cracks. Concrete crushed at top.
T^5_1	1	Diagonal cracks 9 ins. from center on both sides at bottom towards center at top. Secondary crack running from first crack towards end in plane of rods.
T^5_2	2	First hair-line cracks 3 ins. from center, increased and ran up towards center at top.
T^5_3	2	Same as T^5_2-crack, 4 ins. from center.
T^8_1	2	" " " 2 ins. " "
T^8_2	2	" " " 4 ins. " "
T^8_3	2	" " " 2 ins. " "
U^5_1	2	Sudden failure, by vertical crack 2 ins. from center.
U^8_1	2	Sudden failure, by vertical crack 4 ins. from center.

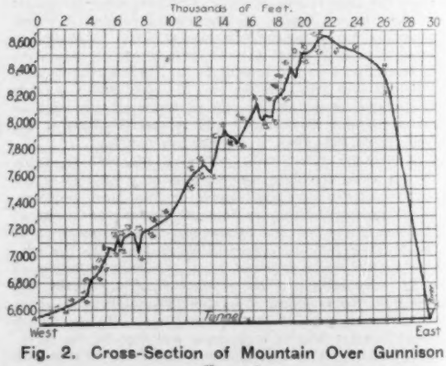


Fig. 2. Cross-Section of Mountain Over Gunnison Tunnel.

*A paper presented before the American Society for Testing Materials, at Atlantic City, N. J., June, 1904.

†Professor of Civil Engineering, University of Pennsylvania, Philadelphia, Pa.

TABLE II.

No. of beam.	Age, days.	Load at which hair-line crack was first noticed, pounds.	Load at which failure crack appeared, pounds.	Ultimate load, pounds.	Deflection at hair line crack, inches.	Deflection at failure crack, inches.
P_1^1	30	Not observed.	10,500	10,500	Not observed.	.1638
P_2^1	29	" "	9,290	9,290	" "	.1485
P_3^1	30	" "	11,720	11,720	" "	.1687
P_4^1	31	" "	8,000	8,125	" "	.3868
P_1^2	30	5,500	7,000	7,500	.2816	.4480
P_2^2	30	5,500- 6,000	6,750	7,280	.2814-.3287	.4684
P_3^2	30	9,500	11,370	11,370	.1409	.1856
J_1^1	30	10,000-10 500	12,000	12,000	.1319-.1438	.1776
J_2^1	30	6,500- 7,000	10,300	10,300	.0743-.0846	.1675
J_3^1	30	10,000-10,500	10,725	10,725	.1475-.1618	.1618
J_4^1	30	4,000- 4,500	9,090	9,160	.1499-.1806	.6415
J_1^2	30	6,000	7,950	8,110	.3359	.4952
J_2^2	30	9,500-10,000	10,430	10,430	.1423-.1628	.1628
R_1^1	30	8,000- 8,500	11,020	11,020	.1036-.1175	.1958
R_2^1	31	7,500	9,000	9,000	.1040	.1671
R_3^1	30	5,500	7,390	7,390	.2935	.4964
R_4^1	30	6,000	8,750	8,780	.2817	.6229
T_1^1	30	6,500	8,000	8,520	.0887	.1895
T_2^1	30	6,500	10,780	11,240	.0768	.2283
T_3^1	30	6,500	10,000	10,180	.0770	.1974
T_1^2	30	5,000- 5,500	6,150	6,370	.2213-.2599	.3933
T_2^2	30	5,500	7,130	7,300	.2512	.4266
T_3^2	30	5,000- 5,500	6,400	6 850	.2270-.2769	.5080
U_1^1	30	.....	.....	1 230	.....	.0063
U_1^2	30	.....	.....	720	.....	.0126

TABLE III.

Beam.	Load at hair-line crack, pounds.	Load at failure crack, pounds.	Ultimate load, pounds.	Deflection at hair line crack, inches.	Deflection at failure crack, inches.
P^3	Not observed.	10,500	10,500	Not observed.	0.1603
J^3	9,400	11,100	11,100	0.1328	0.1731
R^3	8,700	10,150	10,150	0.1281	0.1752
T^3	6,500	9,600	10,000	0.0808	0.2051
P^2	5,750	7,250	7,650	0.3052	0.4344
J^2	5,250	8,500	8,650	0.2583	0.4684
R^2	5,750	8,100	8,100	0.2876	0.5597
T^2	5.500	6,550	6,850	0.2627	0.3424

TABLE IV.—Amount and Properties of Reinforcing Steel.

Type of rod.	Area of Metal—sq. in.	Area of Metal—% of cross section.	Elastic limit.	Ultimate strength.	Modulus of elasticity.	Elongation in 8 ins., %.
P	0.75	1.17	40,500	60,900	30,500,000	23.5
J	0.54	0.84	65,800	102,300	28,500,000	13.5
R	0.76	1.19	58,000	80,500	29,000,000	7.75
T	0.59	0.92	31,900	51,300	28,500,000	13.0

TABLE V.—Elastic limit of Thacher Rods Before and After Molding in Concrete Beam.

The elastic limit was determined by drop-of-beam. The values given are in pounds per square inch.

	Unused rods.	From 8-ft. Beam.—End.	From 8-ft. Beam.—Center.	From 5-ft. Beam.—End.	From 5-ft. Beam.—Center.
	32,030	38,650	40 200	41,710	40,000[1]
	36,450	40,200	41,130	42,930[1]	44,000[2]
	34,640	39,700	41,960	43,920[1]	46,170[1]
Average values	34,370	39,520	41,100	44,850	43,390

[1]Broke in round section. [2]Broke in flat section. All others broke at connection between round and flat section.

and adhesion tests, which will be described in more detail hereafter.

BENDING TESTS.—The beams were supported at the ends on straight, rounded, steel saddles, grooved on the bottom and resting on curved, rounded bearings. The load was applied through a straight, rounded bearing at the center acting on a steel bearing-block 2 ins. wide and 8 ins. long, flat on the bottom and having a rounded groove at the top, curved in a plane normal to the direction of the beam. Thus there was perfect freedom of adjustment at the bearings, and the influence of eccentric loading was practically eliminated. The load was applied to the reinforced beams in increments of 500 lbs. In the case of the two unreinforced beams, the load increment was 250 lbs. for beam U_1^1 and 200 lbs. for beam U_1^2. The deflections were read at the middle of the span on both sides of the beams by means of a pair of micrometer screws reading direct to 0.0001 in., supported in a yoke clamped to the beam in the plane of the rods. These screws made electric contact at the center of a pair of longitudinal steel bars supported by clamp screws attached to a yoke directly above the end bearings and at or about the neutral plane of the beam. These bars were pivoted at one end, and allowed to slide freely on rounded pins at the other. As a matter of fact, no such longitudinal movement could be discerned as the tests progressed. To minimize the chance of error, every reading throughout these tests was made independently by two observers.

In the tables following, the beams are designated by the initial letters of their reinforcement: "P" stands for plain rods, "J" for Johnson, "R" for Ransome, and "T" for Thacher rods. "U" indicates unreinforced beams. The exponents denote the spans in feet and the subscripts the running numbers of the tests in each individual set.

The first indications of failure were in the form of very fine hair-line cracks, transversely across the bottom of the beam for a length of one to two inches. The first to appear were 1 to 4 ins. from the center. On increasing the load others soon appeared at various points across the bottom up to 10 ins. on both sides of the center. These cracks widened and lengthened for a few hundred pounds increment of loading, and then in the majority of cases remained stationary. In the case of beams R_1^4 and R_3^5, for which the load was released after each successive application, these cracks disappeared entirely during the release of the load.

Broadly speaking, final failure occurred in four different characteristic ways designated in the table by the numerals 1, 2, 3 and 4, which may be described briefly as follows:

1. By a diagonal crack starting at the plane of the rods and running up to the center at the top at a mean angle of about 30° with the horizontal.

2. By a vertical crack, starting by the enlargement of a hair-line crack running across the bottom and up the sides, not more than 4 ins. from the center.

3. By a sudden diagonal crack from the center at the top to the end support at the bottom.

4. By crushing at the center at the top of the beam.

Failure occurred by method 1 in 12 cases, by method 2 in 7 cases, by method 3 in 2 cases, and by method 4 in 4 cases.

In cases of failure by method 1 the diagonal crack developed more or less suddenly. In some cases this crack extended to the bottom of the beam and across the same. More commonly, however, it continued, from the point where it reached the plant of the upper surface of the rods, along this plane. In the case of failure by method 4, there were also indications of failure by method 2, but the vertical cracks near the bottom ceased to develop after crushing began at the top.

The manner of failure in each case is described more minutely in Table I. In beams P_3^2 the rods slipped, immediately before final failure. In beams P_1^3 and T_1^3, the adhesion between the rods and the concrete was broken by the crack along the plane of the rods. The rods in these beams slipped in consequence, but after (not before) final failure. In none of the other beams was there any longitudinal movement of the rods. Careful watch was kept on such movement by observing the ends of the rods.

Table II. shows: (1) The loads at which the hair-line cracks were first observed; (2) the loads at which the failure crack became clearly apparent; and (3) the ultimate loads. The deflections corresponding to the first and second loads are also given. The hair-line cracks, which were exceedingly minute and appeared first at the bottom of the beams, escaped detection in beams P_1^3, P_2^3, P_3^3 and P_1^4, which were the first beams tested. In all of the remaining reinforced beams a careful watch was kept on the appearance of these minute fissures. In some cases they first became evident during the application of the load increments. In such cases the values of the load limits between which these cracks became noticeable, are indicated. In all such cases the deflections corresponding to both loads are recorded. It is to be observed that the beams were all tested at the age of 30 days, except beam P_2^3 which was tested at 29 days, and beams P_1^3 and R_2^3, which

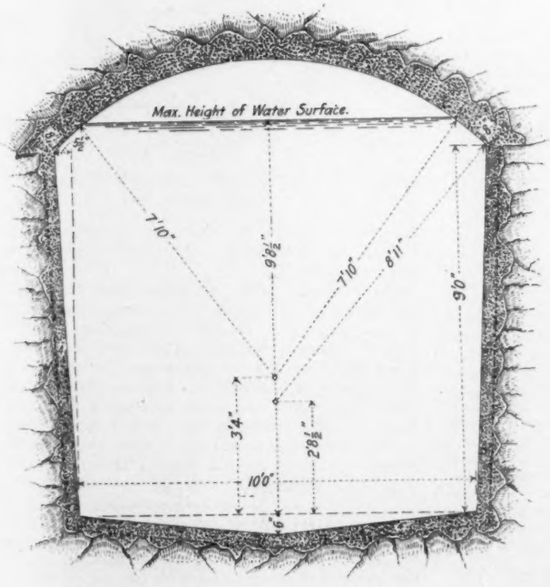


FIG. 3. CROSS-SECTION OF GUNNISON TUNNEL WITH CONCRETE LINING.

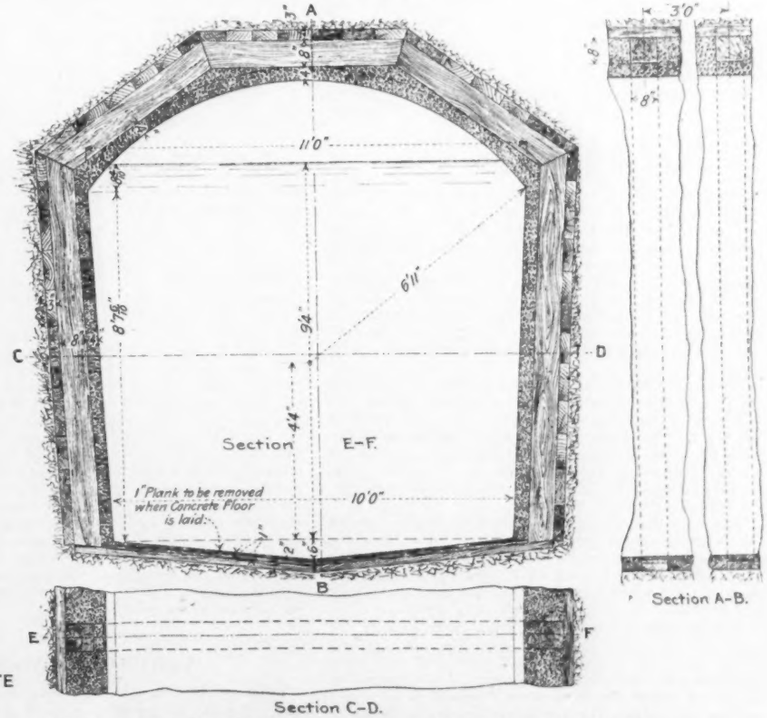


FIG. 4. CROSS-SECTION OF TIMBER LINING FOR GUNNISON TUNNEL.

TABLE VI.—Compression Tests on 6-in. Cubes.

Ultimate Total Loads in Pounds, at 30 days, in Air.

Cubes from different mixtures.	Cubes from single mixture.
78,800	57,000
49,200	54,000
49,100	55,400
52,500	59,000
53,300	60,200
57,900	61,000
90,100	58,000
49,200	54,000
74,200	61,200
.....	50,000
Average values.....61,600	57,000
Average ultimate strength in lbs. per sq. in 1,710	1,580

was tested at 31 days. The influence of the difference of a single day in the age of the three beams mentioned, is regarded as negligible without sensible error.

The moduli of rupture of the two unreinforced beams, computed by the common theory of flexure, were found to be 287 and 250 lbs. per sq. in. for the 8 and 5-ft. beams, respectively, including their own weight, the weight of these two beams being 600 lbs. for the 8-ft. and 380 lbs. for the 5-ft. beam.

The averages of the values in Table II. for the various types of reinforcement and for the two span-lengths used, are shown in Table III. In computing these averages the higher values in columns 3 and 6 of Table II. were used.

Attention has previously been called to the fact that while the rods were nominally ⅜-in. square, there was actually a considerable difference between the smallest lateral dimensions of the rods of the different types. These dimensions as well as the percentage of the metal section to the total cross-section of the beam, are given in Table IV.

The values of the elastic limit in this table represent in reality the values of the yield points; that is to say, these values were fixed by the drop of the scale-beam. The values of the actual elastic limit were somewhat lower. In the case of the Ransome rod there was no well-defined indication of a drop. Each value of the elastic limit, ultimate strength and modulus of elasticity in Table IV. represents the mean of two tests, except the values of the elastic limit and modulus of elasticity of the Ransome rod, which are based on single tests; owing to the twist of this rod it was somewhat difficult to make satisfactory determinations by means of the extensometer. The individual tests constituting each pair showed a fair agreement, especially for the soft steel. The differences were as follows:

Elastic Limit: P, 1,000; J, 3,000; T, 2,600 lbs.

Ultimate Strength: P, 400; J, 6,400; R, 8,400; T, 1,500 lbs.

Elongation in 8 inches: P, 1.0; J, 1.0; R, 0.5; T, 2.0%.

The moduli of elasticity showed a substantial agreement for rods of the same type. It is to be observed, however, that the values of the moduli of elasticity are based on the average unit deformations for the gage-length of 8 ins. For the Johnson rods this is doubtless somewhat less than the unit deformations of the smallest sections, for which the unit stresses were computed; that is to say, the modulus of elasticity of the material itself in these rods is somewhat lower than indicated in Table IV.

Two of the beams reinforced by Thacher rods (T_3 and T_4) were broken open several days after failure and pieces were cut from one end and from the center of each rod. The yield point of these twelve specimens was determined by the drop of the scale-beam, checked by means of dividers. The values thus found, as well as the corresponding values for unused rods of the Thacher type, are shown in Table V.

It is seen that the average value of the elastic limit for these three unused rods is somewhat higher than the average of the two rods in Table IV. The average elastic limit of the five rods is 33,380 pounds. This average, as well as the maximum value, is less than the values found from the rods which had been molded in the beams.

COMPRESSION TESTS.—The compression tests on the concrete for determining the modulus of elasticity were made on four 6 × 6 in. prisms, 24 ins. long 30 days old. Collars were attached to the prisms by means of pointed set-screws making contact at the center of the four sides. The gage-length, center to center of set-screws in the upper and lower collars, was 18.5 ins. An Olsen compressometer, with micrometer reading directly to 0.0001 in. was used for the measurements. The lingers of this instrument were put in contact with vertical extension rods of suitable lengths, attached to the upper and lower collars at diagonally opposite corners. By electric contact in conjunction with a telephone receiver the apparatus was sensitive to one-half of one division, or 0.00005 in.

The modulus of elasticity was sensibly constant for the first three specimens up to a unit stress of about 500 lbs. Its value, in round numbers, for tests Nos. 1 and 2, up to this limit, was 2,000,000 and for test No. 3, 2,300,000 lbs. per sq. in. The diagram for No. 4 was less uniform. The value of the modulus of elasticity based on the average slope up to 600 lbs. per sq. in. was in round numbers, 2,700,000 lbs. per sq. in. The ultimate compressive strength of these prisms was 42,000, 41,500, 46,000 and 47,500 lbs. respectively.

TABLE VII.—Pulling-Out Tests.

Kind of Rod.	Total Load.	Load per lin. inch of rod.	Remarks.
Johnson	13,660	1,138	El. Lim. passed, Conc. cracked.
	12,830	1,069	" " " "
	9,980	832	Concrete cracked.
Plain	6,280	524	Rod pulled out.
	6,190	516	" " "
	5,650	471	" " "
Thacher	10,420	868	Rod broke.
	8,890	741	Concrete cracked.
	9,970	831	Rod broke.
Ransome	22,690	1,891	Concrete cracked.
	16,680	1,390	" "
	19,290	1,608	Rod pulled out.

Compression tests were also made on nine 6-in. cubes, molded from nine different batches of concrete used in making the beams. The resulting values which were somewhat erratic are shown in Table VI. Ten 6-in. cubes were also molded from a single batch of concrete of the same composition used throughout these tests. The results for this series, which were much more uniform, also appear in Table VI.

The surfaces of these cubes were trowel-finished with the greatest care, and the loads were applied through flat metal bearings. The pressure was brought upon the upper plate through a spherical bearing block.

It is to be observed that the same percentage of water (16% of weight of sand and cement) was used for these prisms and cubes as for the beams. Two 6-in. cubes mixed with 10% of water (instead of 16%) and thoroughly rammed, the one placed in water, the other in air for 30 days, developed a compressive strength of over 100,000 lbs., the capacity of the testing machine used.

TENSION TESTS.—The specimens for tensile tests were made of eyebar form. The cross-section was 4 × 4 ins., the width of the heads 6 ins., the distance from center of 1¼-in. pin-hole to back of head was 2½ ins. The gage-length between centers of set-screws through upper and lower collars was 9½ ins. The eyebar heads were made of a rich mortar, reinforced with expanded metal curved half-way around the heads, midway between the pin-hole and the outer periphery. Five specimens were made of plain concrete, and four specimens were reinforced with single central rods of the four types used. In two specimens the heads were reinforced by means of pairs of expanded metal pieces parallel with the plane of the outer flat surfaces of the eyebars. The pins were swiveled in double stirrups held in the grips of the machine. Micrometer readings were taken on opposite sides to 0.0001 in.

In most cases the specimens broke in the body. The readings of the two micrometers were, however, so widely divergent and so erratic and the ultimate strength so low, that it was evident that, through slipping of the grips, and through want of provision for angular adjustment in the plane normal to the face of the bars, failure was brought about through bending. The results are considered valueless, and are not given. It was apparent, however, that the elastic deformation of the bars provided with central reinforcement was considerably lower than that of the plain-concrete bars.

ADHESION OR PULLING-OUT TESTS.—The tests for adhesion were made by imbedding the rods centrally in 6 × 6-in. concrete prisms, 12 ins. long. The rods were imbedded for the entire length of the prisms. The specimens were molded with the rods lying horizontally as in the case of the beams. These tests were also made at the age of 30 days and the results are given in Table VII. The rod was passed through the movable cross-head of the machine and gripped in the upper fixed cross-head. The surface in contact with the lower face of the movable cross-head was bedded in plaster-of-Paris after the rod had been secured in the grips. The plaster was allowed to harden under an initial load of 1,000 pounds. In all cases where the concrete split the contact between the rods and the concrete proved to have been very good. It will be seen that in most cases failure occurred by the breaking of the rods or the cracking of the concrete, thus giving no definite indications of the value of the adhesion, or rather the strength of the bond.

A COLLIMATING GUNSIGHT.

With rapid-fire guns of very costly construction, expensive projectiles and high-power explosives, the importance of rapid and accurate aiming must be generally recognized. A shot costs too much at the present time to be wasted by reason of any personal error in adjusting the sights.

A novel collimating gunsight is being introduced by Messrs. Vickers Sons & Maxim, Ltd., under the name of the "Grubb" sight, and is applicable either to small arms or to small ordnance. The purpose of this sight is to obviate the ordinary difficulty of seeing distinctly at one time both the ordinary foresight and the distant object. By a simple optical contrivance a "ghost" image of a cross, or any other symbol, is seen as if projected upon the object aimed at. The result is that the marksman has only to look at the distant object and to see that the cross is superimposed upon the object. No backsight is required.

The principal features of this sight—as described in the "Journal of the United States Artillery," Vol. XXI., No. 2, are shown in the diagram given herewith. Externally it is a short rectangular metallic tube which is attached to the rear of the gun-barrel, permanently or temporarily, as desired. The rear end of this tube is closed by an inclined piece of plate-glass, silvered on the inner face at the bottom third only. The front end of the tube is closed by a concavo-convex lens, set at an inclination about half that of the rear glass. The concave inner surface of the front glass is coated with a semi-transparent and highly reflective film chemically deposited. Just above the concave glass—in a sort of hood, is inserted a diaphragm made of glass and coated with some opaque coating on which is cut the cross (+) or other sighting device.

In operation, the marksman looks through the tube at the distant object, and sees superimposed upon the distant object the reflected image of the cross.

The operation of sighting is the same as with any other device, and consists simply in bringing the sighting cross into coincidence with the point to be aimed at.

The present device differs, however, from either a plain or a telescopic sight. It forms an image (virtual) of the sighting cross *in the plane of the object* aimed at, so that there is no parallax; nor does the eye require to adjust itself to two different distances. It differs from the telescopic sight in that the rays of light from the *object* pass only parallel surfaces of glass, so that there is no refraction. No image of the object is formed by the device as occurs in a telescopic sight. This

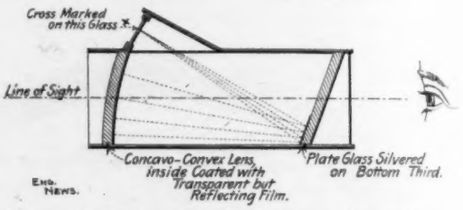

Cross Marked on this Glass X.
Line of Sight
Concavo-Convex Lens, inside Coated with Transparent but Reflecting Film.
Plate Glass Silvered on Bottom Third.
Eng. News.

Diagram Illustrating Optical Principle of the Grubb Gun Sight.

has the advantage that the field of view is not reduced. However, the principle can be applied also to a telescope.

The "Grubb" sight has been fitted to Mauser and other service rifles, to sporting guns and to Maxim guns and similar small ordnance, The sight is adjusted for the various ranges by turning a small disk on top of the sight. This disk is connected with a screw, the threads of which engage with teeth on a segment fixed to the sight bracket. The underside of the disk has a spiral groove which engages with a stud on the range indicator, and this indicator is moved longitudinally by turning the disk.

THE INDIANA STATE FORESTRY RESERVATION was established as a result of a law passed by the Indiana legislature in 1903, authorizing the purchase of a tract of land at a cost of not exceeding $8 per acre, the appointment of a superintendent for the same, and its improvement for the purpose of tree culture and general forestry. The site selected is in Monroe Township, Clark Co. In purchasing this land the State Board of Forestry had several purposes in view. The plan is to cut out all small stunted timber of undesirable species and preserve only those of commercial value. It is also the purpose to establish buildings to be used by summer visitors and students in botanical research. Underbrush, dead and defective growth and unsightly debris were removed from a large area during the summer. Roads have been constructed and all old buildings and fences taken away. On one of the most commanding elevations, Hollister Knob, are the administration buildings, and resident headquarters for the custodian. This knob located near the western boundary of the reservation, has an altitude of about 300 ft. above the mainland of the reserve. One of the first acts of the Board of Forestry was to establish the boundary lines more definitely and to make a topographical map of the reservation. The survey was made by students from the Civil Engineering Department of Purdue University, and the above particulars are taken from a short paper by Mr. R. V. Brewster in the "Proceedings" of the Purdue Society of Civil Engineers.

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ADVERTISING RATES will be furnished upon application. "Want" and "For Sale" notices (special rates), see pages 26-27. The pages for new "Want" "Proposal" and "For Sale" advertisements are held open until Wednesday noon.

The grade crossings of electric railways with other railways and with the public roads are becoming a greater source of danger and a more frequent cause of grave disasters with every passing year. Not only is the number of such death traps rapidly increasing but the rapid increase in speed on electric lines and the growth in traffic and train service multiplies the periods of daily danger at every such crossing.

In Indiana, where electric interurban railways have had a large development within the past few years, there are now* 151 grade crossings of steam railway and interurban electric railway tracks, and the total number of tracks crossed is 232. This is an average of one grade crossing of a double track railway every five miles. Of these crossings, 116 are in towns and cities and 35 are in the country. The making of these grade crossings has been almost invariably opposed by the steam railway companies, whose officers have appreciated far better than the public authorities the great danger which has thus been created. Their efforts have resulted, however, in only 24 crossings made over or under grade, as compared with the 151 grade crossings above mentioned.

As evidence of the torpid state of public opinion respecting these grade crossings, it may be noted that only a single one of them is protected by a complete interlocking plant, although 22 others have hand-operated derailing switches which have to be closed by the electric car conductor from the opposite side of the steam railway tracks before the electric car can cross.

As our readers are aware, the steam railways of Chicago have spent great sums of money in elevating certain parts of their lines and eliminating grade crossings, yet there still remain many such crossings in that city which are the scene of frequent accidents. Recently the accidents at crossings of steam and electric railways have become so numerous in and around Chicago and throughout Illinois that the city authorities of Chicago and the State Board of Railroad Commissioners have taken action. The city has passed orders requiring that all such crossings must be equipped with gates, and that street cars must come to a stop 200 ft. from the crossing. A proposition to require the use of derailing switches in the street railway tracks is also under consideration. The Board of Railroad Commissioners has ordered two meetings, one at Chicago and one at Springfield, which the managers of steam railways, street railways and interurban electric railways are called upon to attend to discuss the grade crossing question.

The electric railways have claimed that they are not within the jurisdiction of the law relating to grade crossing protection, for the reason that the law was passed before such railways existed. In one case the court has held that electric lines are "railways" within the meaning of the law, but no such decision has been finally laid down. It is to be hoped, however, that the courts will finally sustain the position taken by the Illinois Railroad Commission, that all railway grade crossings are under its authority.

*These figures are from a paper by Robert P. Woods, of Indianapolis, read at the last annual meeting of the Indiana Engineering Society.

The latest great electric railway grade crossing disaster to be reported occurred after the preceding note was in type. We refer to the wreck of a street car at a St. Louis grade crossing by a Wabash World's Fair shuttle train, on the afternoon of Sept. 3. The car contained some 50 persons, of whom 7 were killed outright and 19 received serious injuries. The crossing was guarded by two watchmen, one of whom had a flag and the other a bell, but apparently there were no crossing gates, or else they were not lowered. The street railway rules required the car to stop 15 ft. from the crossing and the conductor to go ahead of the car and see that the track is clear and signal the motorman to cross. From the newspaper reports it appears that the car stopped at the crossing and then started up, and stopped directly on the track close in front of the approaching passenger train. It was stated by passengers that the conductor did not get off the car but said to the motorman: "Go ahead, I guess it's all right." The motorman may also have mistaken the signal of the flagman on the crossing.

It is worthy of record that a previous fatal disaster occurred at this same crossing on Jan. 19, 1892, when a large sleigh was struck on the crossing and 8 of its occupants were killed. Only a year ago another accident was narrowly averted at this same crossing. The gates were down and a train was approaching when an electric car crashed through the gates and crossed the track just in time to escape the engine pilot.

Electric power transmission over long distances has been steadily advancing during the past few years with constant new records of greater and greater distances, and the use of higher and higher potential. The advance in commercial work from twenty to two hundred miles, and from 10,000 to 60,000 volts, has been achieved within less than a decade. The amount of work on the design of electrical details which that advance has involved is not generally realized by the outsider, nor even by such electrical engineers as are not in close touch with high-tension problems. In a general way, the difficulties have included power factor, regulation and resonance, switches, safety devices and what not; but first, last and all the time has been the matter of insulation, and this still remains the point of greatest difficulty. The many thousands of porcelain or glass line insulators, all of which must remain in perfect condition in spite of moisture, dust and bullets, constitute a very complex problem to the electric transmission engineers

A paper which is reprinted on another page of this issue gives some suggestion of these difficulties and also brings out clearly another difficulty which promises to limit the increase in voltage in transmission, as those who work in this field will recognize—the brush and glow discharge from the line wire itself. What this brush discharge involves is, first, a large power loss, and, second, a great strain on the insulators.

It will be noted that the line in question was stressed to but 40,000 volts from each wire to ground, which is not a remarkably high voltage; yet the insulators failed quite rapidly, though the line was only 16 miles long and the number of insulators, therefore, relatively small. The power loss is equally important, and it increases very rapidly with slight additional increase in voltage, so that it becomes economically essential to work a transmission line at a normal voltage below the brush discharge limit. While this limit rises somewhat as the size of the conductors is increased, it yet sets fairly definite bounds to the raising of transmission voltages beyond their present point. Whether special means may yet be found for removing this limitation remains for the future to determine.

The communicable character of pneumonia and the susceptibility to the disease of those subjected to exposure in bad weather render significant to engineers and contractors the high mortality from that disease which has been reported of late from various parts of the country. The disease is by no means confined to the winter months, and, as is known to the sorrow of many, it attacks the strong as well as the weak. Crowded quarters promote the spread of the disease, through lessening the resisting power of the individual to the inception and to the attack of pneumonia and by greatly facilitating the spread of the disease from person to person. The moral of these facts, as applied to engineers and contractors, and particularly to those in or responsible for contractors' camps, is to avoid overcrowded and poorly ventilated quarters for themselves and for their men, and to enforce proper measures of isolation upon all persons suffering from pneumonia.

Garbage disposal at Chicago continues to be carried on in the same old shiftless way that has been followed for years. Writing on this topic, the Chicago "Tribune" recently said:

> The present method of removing garbage by dumping it out on vacant tracts of land within sight and smell of the people of the adjoining neighborhoods gives Chicago too strong a resemblance to an African village. There have been three or four aldermanic tours through eastern cities for the purpose of collecting garbage ideas. When the first tour was over we had as many ideas as we have now. If by a year from now we haven't put some of those ideas into operation we shall deserve to have an application made on our behalf to the charity of some philanthropic millionaire.

But why should Chicago, or any other city for that matter, expect any valuable practical results from junketing excursions of aldermen to visit engineering works of whose merits or demerits they are incompetent to judge? Would a private corporation send its Board of Directors to study a complicated technical art, or would it seek the advice of experts skilled in that art? If Chicago wants a decent system of garbage disposal it should treat the problem exactly as it has the problems of sewerage and water supply, and refer the matter to some engineer competent to decide as to the system which it would be most advantageous for the city to adopt.

NEEDED IMPROVEMENTS IN RAILWAY TRACK AND MAINTENANCE OF WAY.

There has long been a tendency in this country, particularly among writers for popular periodicals, to draw comparisons between American and European railways, to the disadvantage of the latter. However satisfying such comparisons may be to our national pride, they are not particularly helpful to our practice. With much greater profit we might imitate the practice of the foreign railway companies which have sent their engineers here to study American methods of railroading, and send our own engineers to Europe to see what they might learn from the methods of working in force there.

American railways have undoubtedly been built very much more cheaply and quickly than those of any other country; but if we consider the condition of large portions of many of our lines which now carry a heavy traffic, and the large annual expenses for maintenance work, perhaps we should find less satisfaction from the comparison. It was real economy to build our railways cheaply in the first place, for they were generally built in advance of the traffic to develop new country. But the conditions have changed, and many main lines which have become crowded with traffic have not yet been rebuilt to bring them in proper relation to the operating conditions. Timber trestles, sliding slopes, unprotected side-tracks, and single track worked

beyond its economical capacity (and without the block system), are features common upon even the most important lines. It is often said that the financial condition of the companies forbids more expensive and substantial work; but let us not forget the peculiar financial transactions which have marred the history of some of our great companies, and which have been the real cause for inadequate expenditures on permanent way.

There is one point in particular wherein American railways show at a disadvantage, in comparison with European roads. We refer to the track which carries our huge freight locomotives with their long and heavy trains, and our fast passenger trains with their enormously heavy cars. Although many improvements have been made within recent years, there is no doubt that the track of many of our main lines still compares unfavorably with that of leading European railways in both construction and maintenance, especially in view of the character of the equipment and traffic. There are undoubtedly many individual stretches of track well built and maintained, but, taking the lines generally, it may be safely said that American railway track is below the standard that should be attained, and entirely out of proportion to the weight of rolling stock and volume of traffic which it carries. Weights and speeds and trains have been increased, without corresponding increase in the strength and stability of the track.

The trouble is not due so much to the appropriations and expenditures for track work being insufficient, as it is to the policy under which these appropriations are made and expended, owing to a failure to realize the governing economic conditions. In hundreds of cases, money is spent over and over again upon the same piece of work or the same stretch of track, without any permanent improvement being obtained, and as a result such expenditures must be continued indefinitely. The principle should be more clearly and generally recognized that expenditures on radical or permanent improvements may effect important economies in maintenance and in other departments of the service.

Of the shape of our rails we have reason to be proud, since the section now most generally used is the result of long and careful expert investigation, while extended experience has proved its merits. The weight of rail is, as a rule, sufficient for the character of the traffic, but the quality of the steel is frequently unsatisfactory to the engineers who have to lay it and care for it. Even the best and heaviest rails, however, are too frequently laid on a defective sub-structure, with old and uneven ties and loose ballast, the track not being designed, built and maintained as one homogenous structure.

At the St. Louis Exhibition there is an extensive display of locomotives and cars of all kinds, but the American track is represented mainly by a limited and very ordinary display of patented joints, frogs and switches, and minor accessories, exhibited by manufacturers and not by railways. In marked contrast to this exhibit is the very complete and interesting railway exhibit of the German Government railways, which is an object lesson worthy of careful study. The rails are not especially heavy, 86 lbs. per yd., but instead of representing rails simply laid upon and tacked to a miscellaneous collection of good, bad and indifferent ties, they represent the wearing surface of a track structure, and it is in this respect that the track challenges comparison with American track. There are three sections of double track, laid partly with steel ties and partly with wooden ties, all the latter being treated by a preservative process. Strong tie-plates are placed between the rails and the ties, which are held together by bolts for the steel, and screw spikes for the wooden ties. Some of the screw spikes are fitted to wooden dowels or sleeves, which are inserted in the tie. The splice bars are well shaped and well-fitted to the rail, cleanly cut in the flanges to fit the tie plates, and stiffened by webs projecting below the rail flange. Creeper plates are placed at intervals, being carefully shaped to fit the tie-plates. The whole construction is substantial and workmanlike.

In the frogs and switches also there is marked difference from American practice. The switch rails, instead of being planed down from ordinary rails and having long, thin tongues, are of specially rolled section, almost rectangular (even to the extreme point), with a fin or rib on the top to fit against and partly beneath the rail head. This rail requires no such reinforcing as is sometimes applied to the long, thin, planed switch rail, and it would be practically impossible to spring it out of shape. The throw of the switches is unusually great. At each switch a long steel plate is placed on the ties, under the stock and switch rails, and a similar plate is put under the frog, which is secured to it by the screw spikes. In American practice the slide plates are usually quite thin, and the sliding surfaces stamped but slightly above the level of the plate, so that a crust of oil and dirt soon forms level with the sliding surface. In this German track, however, the plates themselves are heavier, and the sliding faces are raised very much higher, so that they are less liable to be encroached upon by dust and dirt.

We do not say that this character of track is to be found on every railway in Germany, but from what we have heard and seen we think it safe to say that it is used on all important main lines. And it must be borne in mind that while this represents German practice only, the railways of other European countries present the same conditions. The engineer interested in track construction may spend half an hour with profit in the Austrian Building, at the Exhibition, examining the volumes of drawings of standard track construction; and there are other interesting exhibits relative to British, French and Belgian track construction. Of course, the improvement of track does not lie entirely or mainly with the engineer, but he is the man who should comprehend the situation and be able to impress upon his superiors the advantages and economies of better track construction.

There is one other point to be considered in this connection, and that is the question of maintenance. Here, again, the American railways do not show to advantage. Whatever may be the conditions in individual districts, the average condition is that the work is continuous and pressing, must be done over and over again, and is a constant drain in the way of expenses. The cleaning and reconstruction of ditches in cuts with improperly trimmed or unprotected slopes is one of the most frequent, troublesome and expensive items in this class of work. In many cases it would be a matter of evident economy to take up the problem in earnest, discover the proper remedy, and carry out such work as will permanently stop this trouble. But this means a definite and large individual expenditure, and the company considers that it cannot afford this. As a result, the continual but unnoticed drain of expense in "maintenance" and temporary repairs is allowed to continue.

There appears to be very little definite knowledge of the cost of maintenance of way on individual railways, and in response to recent inquiries in this matter several engineers have frankly stated that no figures are available except those of the annual reports. These reports give (outside the operating expenses) annual expenditures for "maintenance of way and structures," which is sometimes subdivided into rail renewals, roadway, repairs, etc. From this may be ascertained the average expenditure per mile for the entire road; but the amount of labor represented by this expenditure is an unknown quantity. Following this method, however, we obtain the following figures as the average of six or eight leading lines in different parts of the country:

	Eastern States.	Middle States.	Western States.
Maintenance of way:			
Per mile of road	$1,747	$830	$724
Per mile of track	1,175	542	547
Maintenance of way and structures:			
Per mile of road	2,381	1,234	1,020
Per mile of track	1,593	745	759

As to the maintenance force, the work is largely in the hands of the foremen and their gangs, and the best men are continually leaving, their places being filled with green men, who have to pick up their knowledge as best they can, and at the expense of the railway company. This is due largely to the low scale of wages, and also to the common practice of keeping the track forces normally down to the minimum and increasing them temporarily when any extensive work is to be done. The employment of foreign labor of the cheapest class is also a matter to be reckoned with in regard to efficiency and economy of work accomplished. In Europe, the work appears to be planned and executed with more regard to future and permanent results, there is more skilled supervision of the maintenance work, and the section men engage permanently in this class of labor. Thus with a more substantial construction, a more far-sighted policy of maintenance, and a more efficient grade of labor, the maintenance should be more economical. No figures are available for comparison as to the direct cost of this class of work, but it is easy to see that maintenance of way should cost less and show better results under these conditions, than when carried out under a "piecemeal" or temporising policy by a force of men who are continually changing.

With all allowance for individual cases of excellence and for improvements made, it must be acknowledged that the track, with its construction and maintenance, is one of the weak spots of the American railway system. It seems, therefore, that, as already remarked, American railways might well adopt a broader policy in the administration of this department, and might obtain valuable information (pointing the way to important results) by sending their engineers to investigate European practice in track construction and equipment and in the work of maintenance of way.

LETTERS TO THE EDITOR.

Standards Adopted by Manufacturers of Hydrated Lime.

Sir: The majority of the hydrated lime manufacturers of the United States met at Detroit in July of this year, and, among other things considered, it was decided to adopt a general standard for bagging and selling this product. Conforming with this understanding we have issued the card herewith enclosed which shows the standards adopted by the meeting.

This is a steadily growing industry and there are now between 15 and 20 manufacturers of hydrated lime in the United States, with a daily output of over 500 tons, and considering this it seems wise to adopt some general standard to become generally known to the trade, such as exists in the cement industry.

We thought this information might be of interest to many of your subscribers in the contracting line.

Very truly yours, Chas. Warner,
Vice-Pres. Charles Warner Co.

Wilmington, Del., Sept. 1, 1904.

(The standards adopted, which took effect Sept. 1, are as follows—Ed.):

BAGS.—A heavy, closely woven burlap or duck bag, containing 100 lbs., 20 bags to the ton. A paper bag containing 40 lbs., 50 bags to the ton.

QUOTATIONS.—All quotations are made including the cost of the package, no bulk quotations being made.

RETURNED SACKS.—The burlap or duck bags will be repurchased from the customer at ten cents each, when returned to the mill in good condition, freight prepaid.

TERMS OF SETTLEMENT.—A discount of 1% will be allowed for cash in ten days, the discount to be taken on the full price including the bags, f. o. b. manufacturer's plant or shipping point. Net cash 30 days.

The Value of Fine Grinding of Cement.

Sir: The writer is much interested in the article on page 112 in your issue of Aug. 4, in reference to fine grinding of cement.

You have apparently overlooked the fact that some months ago you published an article on the Edison Cement Mill, in which one of the principal points involved was the fine grinding of cement by special machinery.

We find after testing all well-known brands of cement that the fine grinding results in a very marked increase of strength as time progresses. While a 7-day test of finely ground cement may not show a marked difference as compared with a cement ground to ordinary fineness, it has been demonstrated that the strength increases with age, and the percentage of increased strength is very marked. In a recent test where the Edison brand was tested in comparison with three of the highest grades of Portland cement on the market, it was demonstrated that the average of the other three cements showed an increased strength in 28 days as compared with 7, of an average of 17%, while the increased strength from the

Edison brand was 71%. We believe that a large part of this advantage is due to the fine grinding.

Your correspondents are correct in stating that fine grinding decreases output and increases cost, and this will no doubt always be the case with any kind of machinery. We have the same experience at the Edison plant, where the output at 75% through 200 mesh would be about one-third greater than with our present standard of 85% through 200, but we have so clearly demonstrated the superiority of the finely ground cement that we have decided to maintain our standard at 85%, feeling quite sure that the engineers know what they are about in requiring the greater fineness, although it is easy to understand why the cement mills resist the necessity of meeting such stringent requirements so long as present standards will pass muster.

Yours very truly, W. S. Pilling,
Treasurer, The Edison Portland Cement Co.
Girard Trust Bldg., Philadelphia, Aug. 12, 1904.

The Grand Trunk Pacific Railway and American Engineers.

Sir: In reference to the subject discussed in your editorial of Aug. 4 and various letters contained in your issue of Aug. 25 relating to the employment of American engineers on the surveys of the Grand Trunk Pacific allow me to state the following facts:

The original scheme of the Grand Trunk Pacific was the building of a railway from North Bay, Gravenhurst, or some other point in western Ontario to the Pacific at or near Port Simpson; this project was enlarged by influences outside of the Grand Trunk management to its present proportion, that is the building of a trunk line railway from Moncton, New Brunswick, to a port on the Pacific, presumably Port Simpson. The original scheme was by no means a national undertaking.

To undertake its direction Mr. Hays engaged Mr. J. R. Stephens as Chief Assistant Engineer of the Grand Trunk Ry. upon the recommendation of Mr. J. W. Kendrick, 3d Vice-President of the Atchison, Topeka & Santa Fe R. R., and himself an able engineer. Such a recommendation would be deemed sufficient by any railroad manager whether the person were a member of any recognized engineering society or not.

Mr. Stephens in turn offered positions as division engineers to such of his acquaintances as were personally known to him to be able to fill the places. There were only three such positions to be filled, and these were offered to gentlemen irrespective of their nationality. One of those who declined was a Canadian; one who accepted was a native-born Canadian.

I do not know what the policy was on the Middle and Western divisions, as no reliance can be placed in the frenzied, hysterical and distorted reports of the public press, but on the eastern division preference was always given to Canadians or British subjects, and out of 150 persons employed on this division only four are American citizens, or about 2¾%. As regards compensation, there is no difference in the pay of similar positions on account of nationality, but for the reason of prior experience.

The Kyle letter, which really constitutes the main and most offensive incident of the investigation, is regrettable and inexcusable; but it voices only the individual opinion of a man who has had small or no experience with Canadian engineers, and therefore was in no position to either give an opinion or form a judgment. This is the most charitable view one can take of it. The writer has come in direct contact with many Canadian and American engineers and knows of no distinction as to their comparative merits.

As regards the alien labor law, there is no doubt that the framers of the law had in mind. "manual labor" when using the term labor and never intended it to apply to and to include professional services of any kind. By an act passed in the United States in 1893, I believe, professional men, including engineers were exempted from the provision of the law. As the Canadian law is only operative against countries that exclude Canadians, it is questionable whether American engineers, entering Canada under contract, could be legally deported. So far no one has been officially deported; in the only case where such an attempt was made, a stay of proceedings has been granted.

To my knowledge it is the prevalent custom in railway service on both sides of the International boundry to hire such men as appear fit to fill vacancies wherever they can be found irrespective of nationality. A large number of my personal friends and acquaintances, all native-born Canadians have accepted and hold positions in the United States under exactly the same conditions and circumstances as the American engineers on the Grand Trunk Pacific. In fact, the United States has been the Mecca and medium of many Canadian engineers; 10% of the active members of the Canadian Society of Engineers are residents of the United States.

If the Canadian Alien Labor Law is really effective against professional men, than it should be also enforced against actors, and any alien member of a theatrical troop entering Canada under contract or prior arrangement should also be excluded, there can be no discrimination as to professions.

If the building of the Grand Trunk Pacific Railway is strictly a national enterprise, in the same sense as the completion of the Panama Canal will be for the United States, then the engineering part should be taken out of politics and appointments should be made by competitive civil service examination under conditions similar to those on national and state works in the United States and not by political pull.

In the mean time a few Americans have entered Canada and are doing work for the development of the country, the same as Sir William Van Horne, Sir Thomas Shaughnessy and other able Americans who also were "aliens pure and simple," when they entered the country; several of the new-comers had acquired property and built homes for themselves and families prior to becoming citizens, when mainly by the agitation of the Dominion Institute of Amalgamated Engineering for British Columbia (an institute which applied for close corporative powers in the Manitoba legislature about two years ago, but had their application refused on the ground that the Canadian Society of Civil Engineers already covered the field) an investigation was instigated.

This investigation made mountains out of mole hills and was all at variance with the general policy of the Dominion, which is to attract desirable immigration. An absurd, ridiculous and impenetrable labor law principally aimed against Americans and which would have done credit to some pig-tailed hermit nation was introduced in Parliament and passed by the lower house, but shelved by the Senate.

Now that the investigation is over we wonder what it was all about, but perhaps the investigator feels like the old soldier of the battle of Blenheim and can say to an admiring constituency: "It was a famous victory!"

An American Engineer in Canada and M. Can. Soc. C. E.
Aug. 29, 1904.

A PROBLEM IN LOCATING TURNOUTS.

By C. M. Kurtz.*

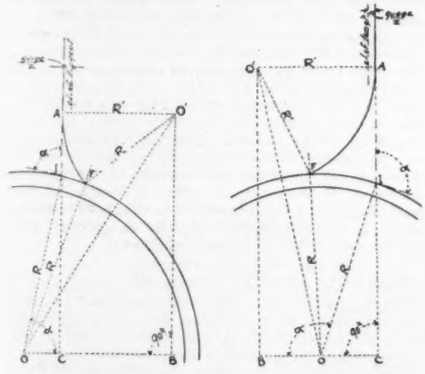
Existing field books give methods for connecting a curve and a tangent to another curve, but these rarely prove practicable in ordinary work in laying out tracks, on account of the entire disregard of the fact that one must use a frog of a certain angle of which the roadmaster has a supply for such work. In applying the problems as found in the field books where a turnout is to form a connection with an established tangent, the engineer is restricted to the use of but one curve, the degree of which is that of the theoretical curve of the turnout, which equals the degree of curve of turnout from a straight track plus or minus the degree of the curve of the track from which the turnout springs. The use of this one curve will rarely prove desirable on account of its being either too sharp or too flat.

As shown in the accompanying figures, four general cases of the problem present themselves. The necessary field work in each case consists in measuring the angle between the center line of tangent and the tangent to the center line of the curve at the point of intersection; or better yet, the angle between a line offset one-half the gage, and the rail which will pass through the frog, as the latter will simplify the problem, and the radius of the rail is determined simultaneously by measuring the middle ordinates for one or more chords.

Having the angle of intersection as described above, the radius of the curved track, and the angle of the frog to be used, and having selected a radius for the connecting curve, the essentials of the location are—the distance between the point of intersection and the point of frog, the central angle of the connecting curve and the distance between the point of intersection and the B. C. of connecting curve.

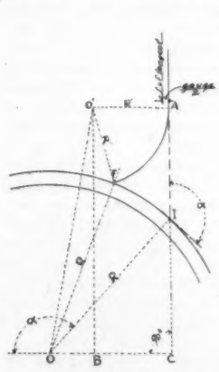
The following solutions offer ready and accurate methods of obtaining the above-named points:

In the figures, let C A represent a line offset one-half the gage from the center line tangent, which is to be connected with the gage line of the rail of the curved track by means of a frog having an angle of F and a curve whose radius is R'. O and O' are the centers, I the point of intersection where the angle α has been measured, and the other notation as shown in the figures. The rail opposite the frog, and the switch rails are not shown as they have no bearing upon the problem. Required to determine the angle I O F.

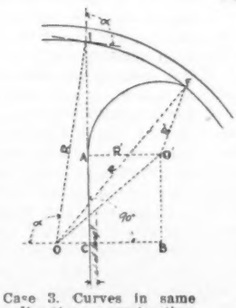


Case 1. Curves in opposite directions. α less than 90°.

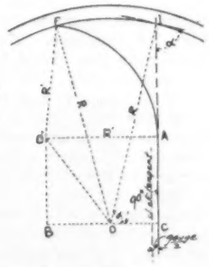
Case 2 a. Curves in opposite directions. α greater than 90°. R' greater than R cos α.



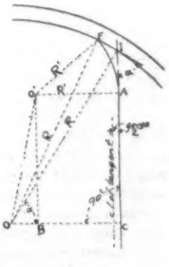
Case 2 b. Curves in opposite directions. α greater than 90°. R' less than R cos α.



Case 3. Curves in same direction. α greater than 90°.



Case 4 a. Curves in same direction. α less than 90° R' greater than R cos α.



Case 4 b. Curves in same direction. α less than 90° R' less than R cos α.

Then measured on the curve,

$$I\,F = R \times \text{angle } I\,O\,F \text{ in minutes} \times \text{a constant } (= 0.00029089), \text{ or Chord } I\,F = 2\,R \times \sin \tfrac{1}{2}\,I\,O\,F.$$

$$\text{In the triangle } O\,O'\,B, \text{ Cos } O'\,O\,B = \frac{O\,B}{O\,O'}$$

O O' is determined by solving the triangle O F O' of which the angle F and its including sides R and R' are known. This also determines the angles F O' O and F O O'.

Cases 1 and 3.	$O\,B = B\,C + O\,C$ $= R' + R \cos \alpha$
Cases 2a and 4a.	$O\,B = B\,C - O\,C$ $= R' - R \cos \alpha$
Cases 2b and 4b.	$O\,B = O\,C - B\,C$ $= R \cos \alpha - R'$

It should then be noted that when O B is computed to be greater than O O', impossible conditions have been imposed; the radius of the connecting curve has been taken too large.

Having determined the angle O' O B, from the figures:

Cases 1 and 2a.	$I\,O\,F = \alpha - (F\,O\,O' + O'\,O\,B)$
Cases 3 and 4a.	$I\,O\,F = 180° - \alpha + F\,O\,O' + O'\,O\,B)$
Case 4b.	$I\,O\,F = O'\,O\,B - (O'\,O\,F + \alpha)$
Case 2b.	$I\,O\,F = \alpha - (180° - O'\,O\,B + F\,O\,O')$
Cases 1 and 2a.	$A\,O'\,F = O'\,O\,B - F\,O'\,O$

*Chief Draftsman, Engineering Department, Assistant Engineer's Office, Southern Pacific Co., Ogden, Utah.

Cases 3 and 4a. $A\,O'\,F = F\,O'\,O - O'\,O\,B$
Case 4b. $A\,O'\,F = F\,O'\,O - (180° - O'\,O\,B)$
Case 2b. $A\,O'\,F = 180° - (O'\,O\,B + F\,O'\,O)$

To determine the distance A I:

Cases 1 and 2. $A\,I = A\,C - I\,C = O'\,B - I\,C$
$= O'\,O \sin O'\,O\,B - R \sin \alpha$

Cases 3 and 4. $A\,I = I\,C - A\,C = R \sin \alpha - O'\,O \sin O'\,O\,B$

The engineer should be careful to add or subtract one-half the gage to the center line radii, as the case may require.

The point A which is the B. C. of the connecting curve, and the point F which is the theoretical point of frog and E. C. of the connecting curve can now be readily fixed in the field and the entire connection staked out.

While the writer realizes that it is better practice to pass the frog end of the connecting curve at the heel of the frog, to impose that condition in the above problem would complicate it beyond measure. In inserting 10 or 15-ft. frogs in a curved track, the trackman cannot help slightly changing the original alinement of the track at that point, and energy spent in making a closer solution than that outlined above will surely be wasted.

ANNUAL MEETING OF THE SOCIETY FOR THE PROMOTION OF ENGINEERING EDUCATION.

By J. L. Van Ornum,* M. Am. Soc. C. E.

The twelfth annual convention of the Society for the Promotion of Engineering Education was held in the lecture hall of the Palace of Education of the Louisiana Purchase Exposition, St. Louis, Missouri, Sept. 1, 2 and 3, 1904. Meetings for the transaction of business and for the reading and discussion of reports and papers, were held mornings and evenings, leaving the afternoons free for the study of the exposition and its exhibits. The number of members present ranks this convention among the best attended ones of the society.

The reports of the secretary and treasurer show the society to be in excellent condition. New members elected during the year just closed increase the membership more than 150%, and the balance in the treasury of the society was increased by a larger percentage than that just given. The loyal interest of the officers and members in the society promises useful work in a field of endeavor which means much to the educational and engineering interests of the country

The presidential address of Professor C. Frank Allen was a scholarly presentation of "The Outlook for Engineering Education." After reviewing the present development of engineering education as it exists in this country, he showed that the number of engineering graduates has increased four times as fast as has the population of the republic, in the last decade, in response to the general demand for trained engineers. From the various circumstances now evident, he concluded that the supply of young men graduating from our engineering colleges will soon exceed the demand for young engineers. It was not thought that the discrepancy would be adjusted by any corresponding reduction in the number of engineering graduates in the future; but rather it is believed that the qualifications of diligence, close reasoning, accurate observation and ready apprehension of the significance of observed facts (which are among the aims of engineering courses) will lead a considerable proportion of the future graduates to find their opportunity as engineering contractors, managers of manufacturing and commercial enterprises, and also to an increasing extent among those who enter upon teaching, journalism and other similar vocations for their life's work.

The five standing committees, on "Entrance Requirements for Engineering Colleges," on "Statistics of Engineering Education," on "Industrial Education," on "Technical Books for Libraries" and on "Requirements for Graduation," all made reports of the investigations and progress they have so far made, and were continued for another year. The last-named committee made an extensive report, giving its opinion of the minimum requirements, in subjects and hours devoted to each, which should be met by an engineering college before granting the first degree in Civil, Mechanical, Electrical or Mining Engineering. After considerable discussion it was agreed that the report covered a field so extensive and with so much detail that the report, printed in full, should be in the hands of each member in order to secure thorough and intelligent discussion; consequently further consideration was postponed to the next annual meeting. It may here be said that, while this investigation does not consider entire uniformity in the engineering courses of the different colleges either attainable or desirable, great advantages will result from adopting a minimum standard for graduation; and from the discussion of the various courses and hours devoted to each course bringing out the diverse opinions upon the value and essentiality of each course. The investigations of the various committees are undoubtedly among the most valuable results of the society's activities.

Thursday evening's meeting was devoted to the general consideration of the preparation and diffusion of knowledge of engineering history and biography. Professor Fletcher's paper on "A Practical Method of Instructing Students in the Biography and History of Their Profession" was a valuable contribution which, like all those read at the convention, must be read in its entirety in order to be appreciated. In brief, it was considered that the history of engineering could be successfully written and would form a valuable addition to engineering literature, while its biography is so extensive and complex that its arrangement would be a very difficult matter. In connection with the latter consideration, it was stated that a "card index," especially of biographical sketches in current engineering literature, would aid in systematizing biographical study. Mr. Waddell's paper, read by Professor Talbot, suggested the publication of a history of engineering by four or five selected authors. Discussion developed the feeling that the success of such publication would depend essentially upon the peculiar adaptation of those selected to work of this kind, and that probably those whose personal qualifications would give promise of successful results would probably be found to be too busy to undertake such a "labor of love." As Mr. Waddell's paper was not available for reference by the special committee on "Proposition for a History of the Engineering Professions," this committee was continued for another year.

Professor Humphrey's paper, Friday morning, considered the important subject of "The Crowding of the Curriculum." He emphasized the desirability of a student's utilizing the three months of the summer vacation in obtaining practical experience in the line of his future professional work, instead of attempting to use these months in study to enlarge his curriculum; he took issue with a recent statement of Mr. Waddell that foreign languages may well give way to other courses; and he advocated the postponement of specialization, for the most part, until after the four-year course. As in the case of most of the papers, the discussion was extensive; and, while it developed various opinions, it was mainly employed in accentuating certain important conclusions of the author.

The papers on "Co-operation in Engineering Investigation," by Professor Talbot; "An Industrial Catalogue Library," by Professor Mack," "Technical Education at the Naval Academy," by Professor Hollis; "Summer School in Mining," by Professor Munroe; "Field Work in Civil Engineering at Iowa State College," by Professor Marston; and "The Preparation of Engineering Text Books," by Mr. Powell, were all valuable contributions in their special lines, and will add materially to our literature upon these subjects. The discussion upon field work for engineering students was especially varied.

Saturday morning Professor Burton described the "Work of the Dean of the Faculty of the Massachusetts Institute of Technology," describing it as comprising the matter of discipline and of forming a distinctive connection between the body of students and the faculty, the general oversight of freshman work, and the position of consulting officer. The need of such an officer in universities having a large attendance in order to retain for them as much as possible of the *personal* quality of instruction and interest that forms one of the advantages of smaller colleges was a great reason for establishing this office. The mathematical papers of Professor Maurer on "Absolute and Gravitational Systems of Dynamic Units," and of Professors Kenyon and Groat on "Teaching Calculus to Engineering Students" indicated the advance that has been made, in recent years, both in teaching mathematical subjects lucidly and pertinently, and in illustrating these subjects continuously by applications to certain practical problems in a way to show the student the direct bearing of the subject upon his subsequent professional studies and his later professional life.

The officers elected to serve during the coming year, were as follows: President, T. W. McNair, President of the Michigan College of Mines; Vice-Presidents, Elwood Mead, University of California, and C. R. Jones, University of West Virginia; Secretary, Milo S. Ketchum, University of Colorado; Treasurer, F. P. Spalding, University of Missouri; Members of the Council, to serve for three years, Professors Thomas Gray, C A. Waldo, A. N. Talbot, J. C. Nagle, W. G. Raymond, L. E. Reber and R. S. Woodward.

*Professor of Civil Engineering, Washington University, St. Louis, Mo.

THE GRAND TRUNK PACIFIC RY. officers are making a transcontinental trip through Canada, visiting the principal cities on the line of the new road and also visiting sites for the Pacific coast terminal and port. At Winnipeg it is probable that the line will enter the large union station now under construction for the Canadian Pacific Ry. It is stated that a Pacific steamship line in connection with the Grand Trunk Pacific Ry. will be operated by the Allan Steamship Co. Included in the party were Mr. Charles M. Hays, President; Mr. Stephens, Engineer of Surveys, and Mr. Hobson, Chief Engineer of the Grand Trunk Ry. The eastern division of the line between Moncton (New Brunswick) and Winnipeg (Manitoba) is to be built under the direction of a special commission, this being one of the requirements of the act as passed by the government. This Transcontinental Railway Commission (whose jurisdiction will include the construction and operation) is composed of the following members: F. B. Wade, of Digby; A. Brunet, of Montreal; R. Reid, of London, and C. A. Young, of Winnipeg. It has as its Chief Engineer Mr. H. D. Lumsden, while the Secretary is Mr. P. E. Ryan, of Ottawa.

AMERICAN MANUFACTURERS OF MINING AND hydraulic machinery and appliances, and machinery and tools used in the glass and ceramic industries are urged by Mr. Julius G. Lay, Consul-General at Barcelona, Spain, to take part in the exhibition of such machinery and tools at Barcelona. The exhibition will open on Sept. 25, 1904. American exhibits will be admitted as late as December, provided applications for space are made by Sept. 25. Up to July 18, Mr. Lay had not heard of a single proposed American exhibit.

A WATER POLLUTION SUIT HAS BEEN WON by the city of Baltimore, Md. The circuit court has enjoined a farmer named Edward A. Cockey from allowing any impure matter to enter drains tributary to the Gunpowder River, which is one of the sources of the water supply of the city.

A BAD RAILWAY ACCIDENT on the Grand Trunk Ry., near Richmond, P. Q., on Aug. 31, caused the death of about ten persons. A west-bound express and an east-bound special excursion train collided because the crew of the special forgot to stop at the meeting-place specified in their orders.

AN ELEVATOR ACCIDENT in Pittsburg, Pa., on Aug. 31, 1904, injured one person fatally and four others less seriously. A passenger elevator in the Park Building was going down from the fourteenth floor when the operator, according to newspaper reports, discovered that he had lost control of the cage. The descent was made with lightning-like rapidity until the first floor was reached, when the safety clutch became effective and the car was stopped before reaching the basement.

The sudden stopping of the car caused injuries to the five passengers as above noted.

AN EXPLOSION OF 800 KEGS OF POWDER occurred Aug. 31 at the works of the Laflin Powder Co. near Punxsutawney, Pa. All the buildings of the plant were wrecked, one man was killed and three badly injured. This is the third explosion at these works in two years.

A NEW ALASKAN CABLE, connecting Skagway, Alaska, with Seattle, Wash., was formally opened for business on Aug. 30, 1904. Commercial cable rates will be materially reduced by the new cable, though press rates are not affected.